PARAMETRIC STUDIES OF INTZE TANK

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

(With Specialization in Building Science and Technology)

By

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CANDIDATE'S DECLARATION

I hereby certify that the work, which is being presented in the dissertation entitled "**Parametric Studies of Intze Tank**" in partial fulfillment of the requirement for the award of the degree of **Master of Technology in Civil Engineering** with specialization in **Building Science & Technology,** submitted in the department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee, is an authentic record of my own work carried out during the period from July 2006 to June 2007 under the guidance **Dr. Vipul Prakash,** Associate Professor and **Dr. Pramod Kumar Gupta**, Assistant Professor, Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Date: 26 June, 2007

Place: Roorkee

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CERTIFICATE

This is to certify that the above statement made by the candidate is true to the best of our knowledge and belief.

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ABSTRACT

Intze tank is an important overhead water storage tank, there for it is necessary that it should be constructed keeping in view its economy. To obtain economical design of tank, the proportion of container such as, staging container diameter ratio, height of cylindrical wall container diameter ratio and horizontal angle of dome have been varied as well as no of column for design of staging .To achieve this objective 6 different capacity of intze tank ranging between 300 KL and 1500 KL has been investigated. For this purpose a computer program in Microsoft excel has been developed .In Microsoft excel program continuity correction have been work out .The design of container is carried out by working stress method but staging is carried out by using limit state method.

To get the economical design of intze tank horizontal angle of conical dome should be less than 45^0, ratio of height of cylindrical wall and container diameter ratio should be between 0.3 and 0.35.On the other hand staging container diameter ratio effects the economy of higher capacity of tank only .As the capacity of tank decreases less than 750 KL its effect diminishes. It is also found that the cost of staging depend upon the number of column.

LIST OF FIGURES

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LIST OF NOTATION

A	Semi central angle of Top dome
a_1	Equivalent radius of cone at top face.
a_2	Equivalent radius of cone at bottom face
A_{gt}	gross area of top ring beam
A_{gm}	gross area of middle ring beam
A_{gb}	gross area of bottom ring beam
Ast	total tension reinforcement
Asc	total compression reinforcement
b_{tr}	width of top ring beam
b_{mr}	width of middle ring beam
b_{rb}	width of bottom ring beam
b_{br}	width of braces
C_f	Force coefficient
D_1	internal diameter of cylindrical dome
D_2	internal diameter of staging
D_{tm}	Mean diameter of top ring beam
D_{wm}	mean diameter of cylindrical wall
D_{mm}	Mean diameter of middle ring beam
D_1c_m	Mean diameter of conical dome at top face
D_2c_m	Mean diameter of conical dome at bottom face
d_{10}	horizontal displacement of member
d_{20}	Angular rotation of member
D_{2bm}	Mean diameter of bottom ring beam
Dc	diameter of column.
d	effective depth

E_s	modulus of elasticity of reinforcement
E_c	modulus of elasticity of concrete
FER	unbalance fixed-edge reaction acting on members
H_c	Rise of conical dome
H_I	cylindrical Height of container excluding free board..
H_{cw}	cylindrical height of container
H_D	height of water above the apex of bottom dome
h_{bd}	height of bottom dome
h_{td}	height of top dome
H	horizontal edge load of cylindrical dome
H_s	height of staging
H_{cl}	clear panel height
H_{cc}	center to center panel height
I_t	moment of inertia of top ring beam
I_m	moment of inertia of middle ring beam
I_b	moment of inertia of bottom ring beam
J	coefficient of lever arm
K_1	probability factor (risk coefficient)
K_2	terrain height and structure size factor
K_3	topography factor
L_c	length of conical portion from origin
L_{br}	length of brace.
m	modular ratio
$M_{\theta b}$	bending moment in bottom beam
M_x	bending moment in middle ring beam due to cantilever portion.
M_t	moment due to torsion
M_u	ultimate moment.
M_e	equivalent bending moment
M_p	joint moment column braces.
M_{br}	hogging moment braces meeting at joint.
N_c	no of column

Np	no of panel
Nb	no of braces
NΦ	meridional stress
Nθ	hoop stress
$N\Phi_w^0$	differentiation of NΦ with respect to angle.
$N\theta_w^0$	differentiation of Nθ with respect to angle
N_X	force along length of cylindrical wall from top face
Ns	force along length of conical dome from top face.
P	% reinforcement
Pu	factored load.
PΦ	force in meridional direction
Pr	force in normal direction
R_{m1}	mean radius of top dome
R_1	inner radius of top dome
R_2	inner radius of bottom dome
R_{m2}	mean radius of bottom dome
R_w	mean radius of cylindrical wall.
R_{tm}	mean radius of top ring beam.
R_{mm}	mean radius of middle ring beam
R_{bm}	mean radius of bottom ring beam
S=	length measured from origin of conical dome in the direction of inclined length of Conical dome
S11	Stiffness of member when horizontal displacement of joint is unity
S22	Stiffness of member when angular rotation of joint is unity
t_{td}	thickness of top dome
t_{tr}	Thickness of top ring beam
t_w	thickness of cylindrical wall
t_{mr}	Thickness of middle ring beam
tc	thickness of conical dome
t_{bd}	thickness of bottom dome

t_{rb}	thickness of bottom ring beam
t_{br}	thickness of bracing.
T_{tb}	Hoop tension in top ring beam
T_{mb}	Hoop tension in middle ring beam
U_1	horizontal displacement of Joint
U_2	Angular rotation of Joint
V	capacity of tank
X_w	vertical distance from top of cylindrical dome
Yc	unit weight of concrete.
Yw	unit weight of water
σ_{ct}	Permissible stress in concrete on water face in direct tension
σ_{cc}	Permissible stress in concrete on water face in direct Compression
σ_{st}	Permissible stress in high yield strength deformed bar
σ_{cb}	Permissible stress in concrete on water face in bending compression
σ_y	characteristic strength of reinforcement.
σ_{ck}	characteristic strength of concrete.
θ	Sámi central angle of bottom dome
α	angle of inclination of conical dome
α_b	semi central angle subtended by supports at the center.
ν	Poisson ratio

CONTENT

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Water is an essential part of life of all living beings. Besides drinking and other day to-day purposes it is also required for fire fighting; industrial and commercial use. The supply of potable water is on of the top priorities of the government to ensure health of its urban and rural population. To ensure continuous supply of water at specified pressures overhead reservoirs are necessary. Intze tank cost ten to twenty percent of the overall cost of the water supply scheme of a town which is quite a substantial amount. Therefore, any saving in the design of Intze tank would lead to economy in the water supply scheme. Investigations are also required to find out safe and economic design and constructional procedures so that scarce constructional materials are used to the optimum limit.

Intze tank consist of a container at the top, supported on a staging to transfer the load of the container to the foundation. Container consists of a domical roof, cylindrical vertical wall, a conical dome and a bottom dome. The staging consists of a frame work of columns and braces or a thin circular shaft. Generally a column-brace system is preferred as staging for Intze tank.

1.2 LITRATURE REVIEW

Elevated water tank are important civil structures, a lot of work has been done on the methods of analysis and design. Many attempts have been made by research workers to optimize the various parameters to obtain economical designs.

Most of the work available on intze tank in existing literature is on the tank where containers supported on column staging. The earlier formulation for analysis and design of Intze tanks based on the membrane and continuity analysis was done by Arya [1].

A computer programmed based on the above analysis of the container was developed by Jain and Singh [2]. A more accurate and tedious analysis has been done using the Finite

Element Method [3]. Work has also been done on Intze tanks supported on circular shafts and various optimal proportions have been recommended by Rao [4, 5]. Sharma [6] has proposed various parameters for Intze tanks stating the percentage share of each member in the total cost of the container. It has been reported that 70% to80% of the cost of the container is due to the cylindrical wall and the conical dome.

Damle and other [7, 8] have tried another shape of an intze tank in which the cylindrical wall has been replaced by a number of segmental cylindrical shells in petal shape. A number of models of circular ground reservoir have been tested and reported to be structurally safe and feasible. It has also been reported that only replacement of vertical wall could yield 20% saving in cost of container .It is further stated that there is potential of 50% reduction in the cost of the container if the conical dome is also modified to a shape consisting of a number of inclined conical conoids.

Singh [9] has analyzed column and bracing staging by stiffness matrix method and compared the results with the tube analogy method applied by Jai Krishna and Jain [11]. Jain and Singh [10] have shown that the tube analogy method gives reasonably accurate design forces apart from being simpler. Rao[12] and Kundoo[13] have also given formulation for analysis of supporting towers acted upon by horizontal forces. Avadesh Kumar [14] has also analyzed R.C.C over head reservoirs and proposed a computer program.

1.3 OBJECT OF STUDY

To judge the correct combination of height and diameter of the vertical cylindrical wall, conical wall and horizontal angle of conical dome is based on economy. There for we try to obtained suitable parameters by which design should be economical for different capacity of intze tank. During design continuity correction have been done at joints. As far as consideration of continuity effects at joints,we know that the solution obtained by membrane theory failed to satisfy continuity of displacement at the junction. We have apply edge loads, consisting of shears and bending moments, in order to correct the discrepancies between the membrane displacements. For that we use bending theory for shell of revolution. For simplicity we assume that the meridian of the dome and cylinder meet at a common point without any eccentricity. For solving continuity problem an Excel program has been developed. Apart from container, the design of the staging is also taken up for the study in this dissertation work.

1.4 SCOPE AND OUTLINE OF STUDY

To get the optimum values of the various parameters of the container and the staging, the parameters were varied as listed in Table 1.1.

This dissertation is divided into 6 chapters. Chapter 1 introduces the problem and in chapters 2, 3 and 4 the analysis and design of the container and staging is covered. In chapter 3 an excel sheet has been developed to over come continuity effect and to calculate redundant forces as well as design the container and staging of capacity 1000 KL. In chapters 5 and 6 the results have been discussed and relevant conclusions have been reported.

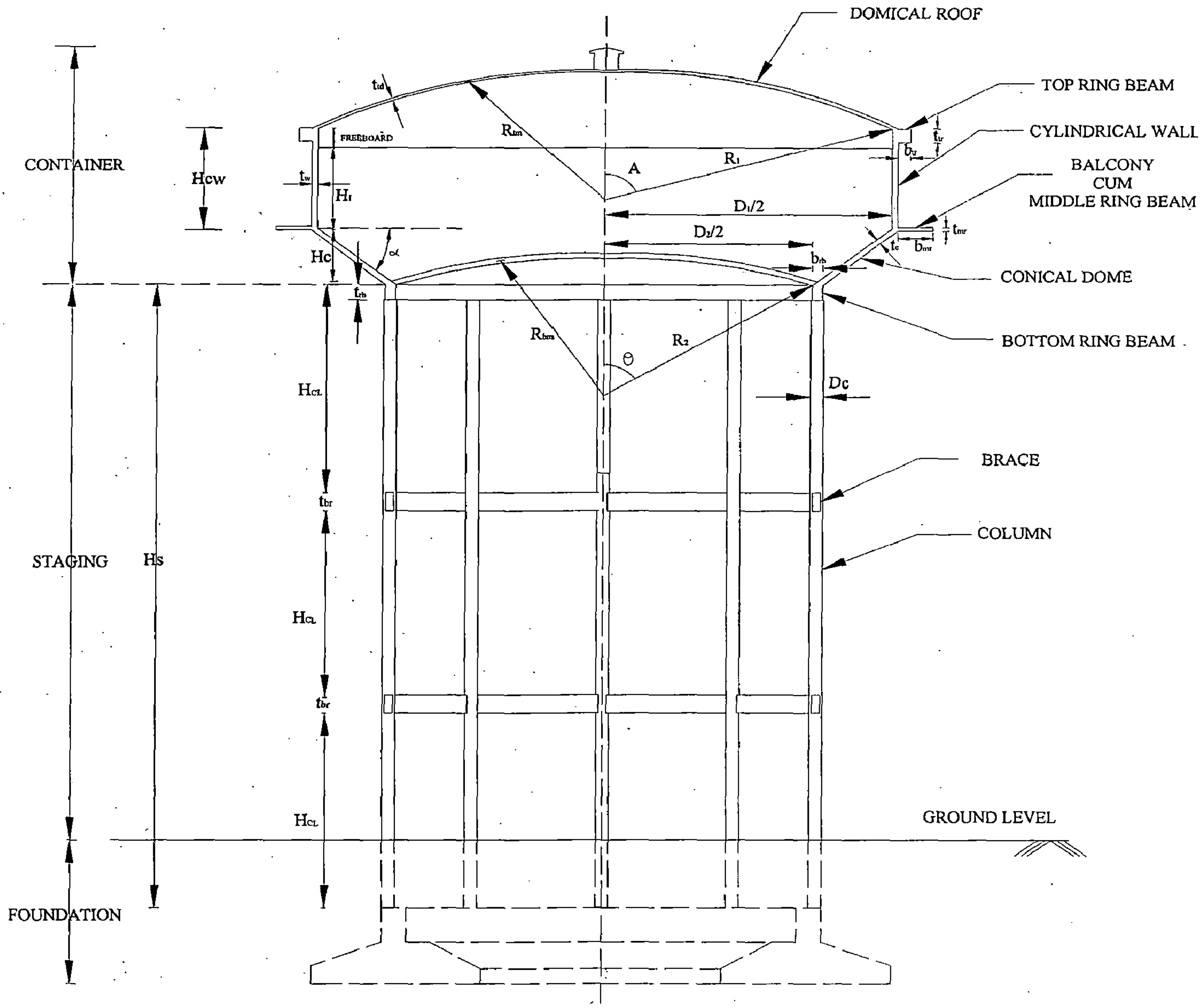

FIG 1.1 TYPICAL DETAIL OF INTZE TANK

Table 1.1 variations of parameters

S.No	Name of parameter	Value of parameter	No. of values of each parameter
1	Basic parameter		
i.	Capacity(V) of container in KL	300,500,750,1000,1250,1500,	6
ii.	Lateral forces a) Wind Zones	Zone 2	1
2	Geometrical parameter		
i.	Height diameter ratio, λ_2	0.20,0.25,0.30,0.35,0.4,0.45,and 0.5	7
ii.	Staging-container diameter ratio λ_1	0.5,0.55,0.6,0.65 and 0.7,and 0.75	6
iii.	Horizontal angle of cone α	35^0,40^0.45^0,50^0,55^0,60^0	6
iv.	Height of staging	15m,18m,21m	3
v.	Number of panels	Minimum	

CHAPTER 2

ANALYSIS AND DESIGN OF CONTAINER

2.1 DIMENSIONS OF C0NTAINER

The capacity of an Intze tank container can be expressed in terms of the volume of the of water is obtained by summation of water containing by cylindrical wall and conical dome and subtracting the volume of bottom dome lies in conical region

V1=volume of water containing by cylindrical wall.

V2= volume of water containing by conical dome

V3= bottom dome lies in conical region

$$V1=\{\Pi*(\frac{D_1}{2})^2*H_w\} \qquad (2.1)$$

$$V2=\{\frac{\Pi}{3}*(\frac{D_1}{2})^2*H_2\}-\{\frac{\Pi}{3}*(\frac{D_2}{2})^2*H_3\} \qquad (2.2)$$

$$V3=\frac{\Pi*R_2^{\ 3}}{3}(2-3*Cos\theta+Cos^3\theta) \qquad (2.3)$$

$$V=V1+V2-V3 \qquad (2.4)$$

$$H_2=\frac{D_1}{2}*Tan\alpha$$

$$H_3=\frac{D_2}{2}*Tan\alpha$$

Let

$$\lambda_1=\frac{D_2}{D_1}$$

$$\lambda_2=\frac{H_w}{D_1}$$

For a given capacity the diameter of container

$$D_1^{\ 3}=\frac{24*V}{\pi\left[6\lambda_2+Tan\alpha(1-\lambda_1^{\ 3})-\frac{\lambda_1^{\ 3}(2-3Cos\theta+Cos^3\theta)}{Sin^3\theta}\right]} \qquad (2.5)$$

Other dimensions

$H_1 = \lambda_2 * D_1$ =cylindrical height of container containing water.

Free board=0.3 m

$H_{cw} = H_1 + 0.3$= cylindrical height of container

$D_2 = \lambda_1 * D_1$ =staging diameter

$h_{td} = \frac{D_1}{5}$ =Rise of top dome

$h_{bd} = \frac{D_2}{5}$ =Rise of bottom dome

$Hc=(H_2-H_3)$= Height of conical dome

$R_1 = \frac{D_1}{2\text{Sin A}}$

$R_2 = \frac{D_2}{2\text{Sin}\theta}$

2.2 TOP DOME

Top dome subjected to live load and dead load. The dome is accessible only for maintenance purposes and hence live load can be taken as 1 KN/m^2 as per IS: 875-1987 [19]. It is supported by top ring beam and cylindrical dome. The rise of top dome (h_{td}) is usually kept 0.2 times the diameter of container D_1.

The Radius of top dome is given by

$$R_{m1} = \frac{D_1}{2\text{SinA}} + \frac{t_{td}}{2} \tag{2.6}$$

The maximum stress occurs at the edge of the dome. The membrane stresses in the dome are

$$N\Phi t_d = -\frac{W_{td} * R_{m1}}{1+CosA} \quad \text{N/m} \tag{2.7}$$

$$N\theta_{td} = -\ W_{td} * R_{m1} * \left(CosA - \frac{1}{1+CosA}\right) \quad \text{N/m} \tag{2.8}$$

Where

W_{td}=(Dead load +live load) per unit area of dome

$$A = \ Tan^{-1}\frac{D_1}{2(R_1 - h_{td})} \tag{2.9}$$

The circumferential stress changes from compressive to tensile if Semi central angle of dome exceeds 51.8 degree.

Hence it is desirable to keep Semi central angle less than 45 degree to avoid double form of shuttering for the shell. The thickness of dome for maximum thrust criterion is

$$t_{td} = \frac{N\phi}{1000 * \sigma_{cc}} \quad \text{mm} \qquad (2.10)$$

Thickness of dome is primary controlled by practical consideration, since the stress is compressive nature and the thickness obtained from equation (2.10) is small. The thickness normally provided is 80 mm. Dome is reinforced by nominal reinforcement of 0.3% mild steel and 0.24% tor steel in the form of square mesh.

2.3 TOP RING BEAM

Top ring beam provided to resist horizontal component of the meridional thrust of top dome. The center line of the ring beam is aligned with center line of the top dome. The hoop tension is given by

$$T_{tb} = \frac{N\phi_{td}\ \mathrm{Cos}A * D_{tm}}{2} \quad \text{kg....} \qquad (2.11)$$

Area of tension reinforcement is given by

$$Ast_2 = \frac{T_{tb}}{\sigma_{st}} \quad \text{cm}^2 \ \ldots\ldots \qquad (2.12)$$

The top ring beam is also designed on no crack basis as to avoid possible corrosion of the reinforcement and also to provide good stiffness to support the roof dome. Thickness of top ring beam is primary controlled by consideration

$$\text{Computed tensile stress } (\sigma t) = \frac{T_{tb}}{b_{tr} * t_{tr} + (m-1) * Ast_2} < \sigma_{ct} \ \ldots\ldots.. \qquad (2.13)$$

σ_{st} =Permissible stress in high yield strength deformed bar

σ_{ct} =Permissible stress in concrete on water face in direct tension

2.4 CYLINDRICAL WALL

For membrane analysis, the wall is assumed free to deform at both edges and thus under a pure hoop tension. Minimum thickness for wall will be taken as 100mm. The effects continuity on wall has been discussed in next chapter (3).

For dead load

W1=Dead load of top ring beam

$=2500*t_{tr}*b_{tr}$ kg/m …… (2.14)

$Nx=(N\phi_{td}*\sin A+W1)-Yc*t_w*X_w$ kg/m ……… (2.15)

Where

Nx= Load on cylindrical wall due to its dead load per unit length

$N\theta=0$

For water load

$Nx=0$

$$N\theta_w=\frac{Yw*D_{wm}*X_w}{2} \quad \text{....kg/m} \qquad (2.16)$$

Where

Xw= vertical distance from top of wall

Area of tension reinforcement is given by

$$Ast_3=\frac{N\theta(avg.)*\Delta x}{\sigma_{st}} \quad \text{cm}^2 \text{ ....} \qquad (2.17)$$

Area of compression reinforcement is given by

$Asc_3=0.24\%\ b_w*t_w$ tor steel.

Where

Δx=zone length

$b_w=100$ cm

The cylindrical wall is also designed on no crack basis. Thickness of cylindrical wall is controlled by consideration

$$\text{Computed tensile stress } (\sigma t)=\frac{N\theta(avg.)*\Delta x}{b_w*t_w+(m-1)*Ast_3}<\sigma_{ct}.$$

2.5 MIDDLE RING BEAM

The purpose of ring beam is to provide a horizontal support to conical wall. It is provided wide enough so as function as a balcony for inspection purposes. The design of ring beam is governed by hoop tension caused by horizontal component of meridonial thrust.

The effects continuity on conical dome will studies in next chapter (3).

The total load on middle ring beam is

Let

W1 =Dead load+Live load of top ring beam ....kg/m

W2 = Nx(at X_w=H_w) ....kg/m

W3 = dead load of middle ring beam.

= ($2500*b_m r*t_{mr}$) kg/m .... (2.18)

We = (W1+W2+W3)

Hoop tension in middle ring beam is given by

$$T_{mb} = \frac{We * \cot\alpha * D_{mm}}{2} \quad kg........ \tag{2.19}$$

Where

α is inclination of conical Dome with horizontal

Area of tension reinforcement is given by

$$Ast_{4a} = \frac{T_{tb}}{\sigma_{st}} \quad cm^2 .... \tag{2.20}$$

Thickness of middle ring beam is primary controlled by consideration

$$\text{Computed tensile stress } (\sigma t) = \frac{T_{mb}}{b_{mr} * t_{mr} + (m-1) * Ast_{4a}} < \sigma_{ct}$$

Live load on the balcony can be taken as 1.5 KN/m^2 as per IS: 875-1987 [19].

Self weight and live load on the balcony ring beam

W_{bc}=($2500*b_m r*t_{mr}+1.50*t_{mr}$) kg/m ..... (2.21)

$$N = \frac{1}{1 + \frac{\sigma_{st}}{m * \sigma cb}} \quad .... \tag{2.22}$$

J=1-N/3

$d = t_{mr} - 25 - \frac{\phi_t}{2}$ ..... mm

$$M_X = \frac{W_{bc} * b^2_{mr}}{2}$$

Area of tension reinforcement in redial direction.

$$Ast_{4b} = \frac{Mx}{\sigma st * j * d} \quad cm^2 .... \qquad (2.23)$$

Where

σ_{st} =Permissible stress in high yield strength deformed bar

σ_{cb}=Permissible stress in concrete on water face in bending compression

2.6 CONICAL DOME

Conical dome support a uniform vertical wall at it top edge. If this dome is assumed as consisting of individual slanting strips, then each strip will tend to rotate outwards about it bottom edge. This will increase the circumference of the circle joining at the top of these strips and each strip will separate out from each other. Since the strips are joined to each other monolithically and can not separate, a hoop tension will created at the top of this dome. This hoop tension will exert a radical inward force at each strip and will oppose it rotation outward. So that the moment about bottom edge of strip, causing rotation is zero. The resultant force must lie along the slant surface of strip. The magnitude of radial force created at top edge is so much that on combining with vertical load the resultant lies along the meridian of conical dome. Thus the vertical load at top edge of the conical dome is supported by it with the creation of meridian thrust and hoop tension. In the same way water pressure on the conical dome and its own weight acting at any point give rise to hoop tension at each plane, whose inward reaction, together with the water pressure and weight of dome, cause a resultant force which is meridoinal.There is no moment and shear in conical dome.

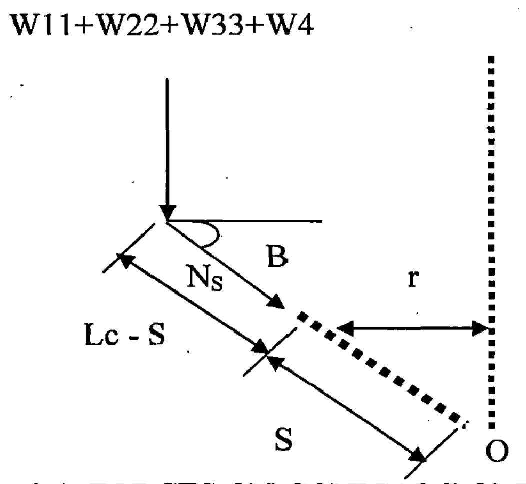

Fig 2.1: FORCES ON CONICAL DOME

For Dead load

Let

W11= weight of W1

$=W1*\Pi*Dt_m$ kg...... (2.24)

W22=weight of W2

$= W2*\Pi*D_{wm}$ kg...... (2.25)

W33= weight of W3

$=W3*\Pi*Dm_m$ kg...... (2.26)

Lc=length of conical portion from origin of conical dome

$$Lc=\frac{Dc_m}{2Cos\alpha} \quad m \;.... \tag{2.27}$$

r=radius at any height Xc from top face conical dome where $Xc \leq Hc$

$$=(\frac{Dc_m}{2})(\frac{LcSin\alpha - Xc}{LcSin\alpha}) \quad m.... \tag{2.28}$$

S=length measured from origin of conical dome in the direction of incline length

$$=Lc-\frac{Xc}{Sin\alpha} \quad m.... \tag{2.29}$$

W4= weight of conical dome at distance" r "as shown in figure

$$= \Pi(\frac{D_{cm}}{2}+r)\sqrt{(Xc^2+(\frac{D_{cm}}{2}-r)^2} \quad \text{kg} \ldots\ldots \qquad (2.30)$$

$$Ns_{Dc} = \frac{W11+W22+W33+W4}{2*\pi*r*Sin\alpha} \quad \text{kg/m} \ldots\ldots \qquad (2.31)$$

$N\theta_{Dc}$=circumferential force in conical dome due to dead load.

$$= Yc*tc*Cos\alpha(Lc*Cot\alpha-(\frac{Xc*Cot\alpha}{Sin\alpha})) \quad \text{kg/m} \ldots.. \qquad (2.32)$$

For Water Load

H_1=height of water above top of cone

Ns_w=force along length of conical dome from top due to water.

$$Ns_w = Yw*Cot\alpha\left[(-\frac{H_1*Lc^2}{2})-(\frac{Lc^3*Sin\alpha}{6*s})+(\frac{H_1+LcSin\alpha)*S}{2})-(\frac{S^2*Sin\alpha}{3})\right]$$

Kg/m...... (2.33)

$N\theta_{wc}$= circumferential force in conical dome due to water

$$N\theta_{wc} = Yw(H_1+LcSin\alpha-S*Sin\alpha)S*Cot\alpha \quad \text{kg/m} \ldots\ldots \qquad (2.34)$$

Area of tension reinforcement is given by

$$Ast_5 = \frac{N\theta(avg.)*\Delta S}{\sigma_{st}} \quad \text{cm}^2 \ldots\ldots \qquad (2.35)$$

Area of compression reinforcement is given by

Asc_5=0.24% b_c*t_c for tor steel.

Where

ΔS=zone length in conical dome

The conical dome is also designed on no crack basis. Thickness of conical dome is controlled by consideration

$$\text{Computed tensile stress } (\sigma t) = \frac{N\theta(avg.)*\Delta S}{b_c*t_c+(m-1)*Ast_5} < \sigma_{ct}.$$

2.7 BOTTOM DOME

The bottom dome is of spherical shape and is subjected to the weight of water over it and self weight. It is supported by a bottom ring beam. The rise of top dome (h_{bd}) is usually kept 0.2 times the diameter of container D_2. The Radius of top dome is given by

$$R_{m2} = \frac{D_2}{2\text{Sin}\theta} + \frac{t_{bd}}{2} \quad \text{m} \ldots\ldots \tag{2.36}$$

The thickness normally provided is 80 mm. The maximum stress occurs at the edge of the dome. The membrane stress resultants in the dome are obtained separately for dead load and water.

For Dead Load

$$N\Phi_{Dbd} = -\frac{W_{bd} * R_{m2}}{1 + Cos\theta} \quad \text{kg/m} \ldots\ldots\ldots\ldots \tag{2.37}$$

$$N\theta_{Dbd} = -\ W_{bd} * R_{m2} * \left(Cos\theta - \frac{1}{1 + Cos\theta} \right) \quad \text{kg/m}\ldots\ldots\ldots \tag{2.38}$$

For Water Load:

H_D=height of water above the apex of bottom dome= (H_1+Hc-h_{bd})

$$N\Phi_{Wbd} = \frac{Yw * R_{m2}}{Sin\theta^2} \left[\left(\frac{H_D + R_{m2}}{2}\right) * (\cos\theta)^2 - \left(\frac{R_{m2}}{3} * (\cos\theta)^3\right) - \frac{H_D}{2} - \frac{R_{m2}}{6} \right] \quad \text{kg/m}\ldots.(2.39)$$

$$N\theta_{Wbd} = -\ Yw * R_{m2}\left[H_D + R_{m2}(1 - \cos\theta)\right] - N\phi_{wbd} \quad \text{kg/m}\ldots.. \tag{2.40}$$

$N\Phi = N\Phi_{Dbd} + N\Phi_{Wbd}$

$N\theta = N\theta_{Dbd} + N\theta_{Wbd}$

Thickness of dome is primary controlled by practical consideration, since the stress is compressive nature and the thickness obtained from equation (2.10) is small, the thickness normally provided is 100 mm.The dome is reinforced by nominal reinforcement of 0.3% mild steel and 0.24% tor steel in the form of square mesh.

2.8 RELEVENT PARAMETER

2.8.1 Volume of concrete

(a) Top Dome

$$V_{tdc}=\frac{2\pi * t_{td} * R_{m1} * h_{td}(2-3CosA+Cos^3A)}{3*(1-CosA)} \quad m^3 \ldots.. \qquad (2.41)$$

(b) Top Ring beam

$$V_{trc}=2\pi * R_{tm} * b_{tr} * t_{tr} \quad m^3\ldots\ldots \qquad (2.42)$$

(c)Cylindrical Wall

$$V_{cwc}=2*\pi * H_{cw} * R_w * t_w \quad m^3\ldots\ldots \qquad (2.43)$$

(d) Middle Ring beam

$$V_{mrc}=2\pi * R_{mm} * b_{mr} * t_{mr} \quad \ldots..m^3 \quad m^3\ldots\ldots \qquad (2.44)$$

(e)Conical Dome

$$V_{cdc}=\frac{\pi * t_c * Sec\alpha}{3}\left(D1_{cm}+D2_{cm}\right) \quad m^3\ldots\ldots \qquad (2.45)$$

(f) Bottom Dome

$$V_{bc}=\frac{2\pi * t_{bd} * R_{m2} * h_{bd}(2-3Cos\theta+Cos^3\theta)}{3*(1-Cos\theta)} \quad m^3\ldots\ldots \qquad (2.46)$$

Total Volume of oncrete

$Vs= V_{tdc} + V_{trc} + V_{cwc} + V_{mrc} + V_{cdc} + V_{bc}$

2.8.2 Volume of reinforcement

(a) Top Dome

$$V_{tds}=0.0048\ V_{tdc} \quad m^3\ldots\ldots \qquad (2.47)$$

(b) Top Ring Beam

$$V_{trs}=\pi * D_{tm} * Ast_2 \quad m^3\ldots\ldots \qquad (2.48)$$

(c) Conical Wall

$$V_{cws}=\pi * D_{wm} * Ast_3 + 0.0024 * V_{cwc} \quad m^3\ldots\ldots \qquad (2.49)$$

(d) Middle Ring beam

$V_{mrs} = \pi * D_{mm} * Ast_{4a} + Ast_{4b} * b_{mr}$ m^3...... (2.50)

(e) Conical Dome

$V_{cds} = 0.0024 * V_{cdc} + \sum 2\pi * r_{mi} * Ast_i$ m^3...... (2.51)

Where

i=no of zonal length in Hc

$\sum Ast_i = Ast_5$

(f) V_{bs}=0.0048 V_{bc} m^3..... (2.52)

Total Volume of reinforcement

$Vs = V_{tds} + V_{trs} + V_{cws} + V_{mrs} + V_{cds} + V_{bs}$ .... (2.53)

2.8.3 Cost of container

On the basis of the present prevailing rates of material the rates of various items are given as under:

(1) Cement concrete of grade M25 in superstructure excluding cost of centering and shuttering — Rs.2900 per cu.m

(2) Tor steel reinforcement including cutting,bending,placement and its wastage — Rs18000 per ton

There for the cost of container =Vc*29000+78.5*18000*Vs Rupees ...... (2.54)

CHAPTER 3

CONTINUTY PROBLEM

3.1 GENRAL

The pure membrane state of stress will exit so long as each shell is simply supported at its edges, that is, it is able to undergo resulting displacement without restrain, while the supports supply the necessary reaction to balance the meridionally forces. This however is not possible in practices and the edge displacement is actually restrained. This give to raise secondary stresses in the form of edge moment and hoop stresses. It will be seen that discontinuity would occur at junction of different shells. There will, therefore come into play additional joint reaction to maintain continuity. It will be seen that the angle at corner of top dome ,cylindrical wall and corner of cylindrical wall and conical dome tend to close resulting in sagging moment, while the angle at corner of conical dome and bottom ring beam tend to increase to hogging moment. Also at the joint the edge of top dome and vertical wall are pulled outward, resulting in tensile hoop stresses, while the top ring beam at that joint is pushed inward and hoop tension reduced. Similarly the middle ring beam and edges of conical dome at joint are pushed inward causing reduction of hoop tension while the vertical wall is pulled outwards with increased hoop tension. At joint of conical dome and bottom dome the conical dome is pushed inward, the bottom dome and bottom ring beam pulled outward. This cause a hoop compression in conical dome and cause of reduction of hoop compression in bottom ring beam and bottom dome.

The complete analysis of intze tank, consist two parts (a) membrane analysis (b) effect of continuity. The net stress are obtained by adding the stresses due to the above two cause. The stresses due to continuity are obtained by applying the principle of consist deformation. the vertical displacement are always consistent at each joint as each shell is free to deform in this direction and consistency has only to be satisfied for horizontal and angular displacement between shell meeting at a joint. This will need of stiffness of each shell at edge for horizontal and angular movements. The stiffness is given by Kelkar V.S and Sewell R.T," Fundamental of the analysis and design of shell structures"

3.2 CONTINUITY CORRECTION

The forces due to continuity are obtained by applying equation of compatibility

$X_i = S_i * U_i + FER_i$ .... (3.1)

Where

i=symbol of Dome, ring beam, cylindrical wall, conical shell

FER=unbalance fixed- edge reaction acting on the members , corresponding to U_1 and U_2, acting on different member.

X1and X2 are the redundant forces of members

For solving U1 and U2 the governing equation is

$$\begin{bmatrix} \sum s11 & \sum s12 \\ \sum s21 & \sum s22 \end{bmatrix} \begin{bmatrix} U_1 \\ U_2 \end{bmatrix} + \begin{bmatrix} \sum FER_1 \\ \sum FER_2 \end{bmatrix} = 0 \quad .... \qquad (3.2)$$

where

$FER_1 = -(S11 * d_{10} + S12 * d_{20})$ ..... (3.3)

$FER_2 = -(S21 * d_{10} + S22 * d_{20})$ ... (3.4)

U_1= horizontal displacement of Joint

U_2= Angular rotation of Joint

d_{10}=horizontal displacement of member

d_{20}=Angular rotation of member

After obtaining U_1 ,U_2 ,FER_1 and FER_2 of each member, redundant forces $X_1$and X_2 of each members obtained by governing equation

$$\begin{bmatrix} x1 \\ x2 \end{bmatrix} = \begin{bmatrix} s11 & s12 \\ s21 & s22 \end{bmatrix} \begin{bmatrix} U1 \\ U2 \end{bmatrix} + \begin{bmatrix} FER1 \\ FER2 \end{bmatrix} \qquad (3.5)$$

ASSUMPTION

(1) Radial thrust

Out ward positive

Inward negative

(2) Moment

Clock wise positive

(looking at the meridian on the left hand Side of the axis of revolution)

Anti clock wise negative

3.2.1 Continuity correction 1: At joint of top dome, cylindrical wall and Top ring beam.

STIFFNESS OF TOP DOME

$$S_{td}=\begin{bmatrix} \dfrac{E*t_{td}}{R_{m1}*\alpha_{1t}*(\sin A)^2} & \dfrac{E*t_{td}}{2\alpha_{1t}^{2}*\sin A} \\ \\ \dfrac{E*t_{td}}{2\alpha_{1t}^{2}*\sin A} & \dfrac{E*t_{td}*R_{m1}}{2\alpha_{1t}^{3}} \end{bmatrix} \tag{3.6}$$

Where

$$E=5000\sqrt{\sigma_{ck}}$$

$$4\alpha_{1t}^{4}=\frac{12(1-v^2)*R_{m1}^{2}}{t_{td}^{2}} \tag{3.7}$$

$$E\Delta H=Ed_{10}=\frac{R_{m1}*SinA(N\theta_{td}'-vN\phi_{td})}{t_{td}} \tag{3.8}$$

$$E\chi=Ed_{20}=\frac{[N\theta_{td}^{0}-vN\phi_{td}^{0}]+\mathrm{CotA}(1+v)(N\theta_{td}-N\phi_{td})}{t_{td}} \tag{3.9}$$

Where

Δ_H= horizontal diflection

χ = angle of rotation

$$N\Phi_{td}=-\frac{W_{td}*R_{m1}}{1+CosA} \tag{3.10}$$

$$N\theta_{td} = -\ W_{td} * R_{m1}\left(CosA - \frac{1}{1+CosA}\right) \tag{3.11}$$

$$N\Phi_{td}{}^{0} = \frac{-W_{td} * R_{m1} * SinA}{(1+CosA)^2} \quad \ldots\ldots \tag{3.12}$$

$$N\theta_{td}{}^{0} = \frac{W_{td} * R_{m1} * SinA * (2+2CosA+CosA^2)}{(1+CosA)^2} \tag{3.13}$$

Where

(θ^0) indicate differentiation

STIFFNESS OF CYLINDRICAL WALL

$$S_w = \begin{bmatrix} \dfrac{E * t_w}{R_w * \alpha_{1w}} & \dfrac{-E * t_w}{2\alpha_{1w}{}^2} \\ \\ \dfrac{-E * t_w}{2\alpha_{1w}{}^2} & \dfrac{E * t_w * R_w}{2\alpha_{1w}{}^3} \end{bmatrix} \tag{3.14}$$

$$E\Delta H = Ed_{10} = \frac{H * R_w{}^3}{2 * (\frac{K_w}{E})\alpha_{1w}{}^3} \tag{3.15}$$

$$E\chi = Ed_{20} = \frac{-H * R_w{}^2}{2 * (\frac{K_w}{E})\alpha_{1w}{}^2} \tag{3.16}$$

Where

$$4\alpha_{1w}{}^4 = \frac{12(1-\nu^2) * R_w{}^2}{t_w{}^2} \tag{3.17}$$

$$K_w = \frac{E * t_w{}^3}{12(1-\nu^2)} \tag{3.18}$$

$$H = N\Phi_{td} * CosA \tag{3.19}$$

STIFFNESS OF TOP RING BEAM

$$S_{tr}=\begin{bmatrix} \frac{E*A_{gt}}{R_{tm}^{2}} & 0 \\ 0 & \frac{E*I_t}{R_{tm}^{2}} \end{bmatrix} \tag{3.20}$$

For top ring beam

$$E\Delta H = Ed_{10} = \frac{T_{tb} * R_{tm}^{2}}{A_{gt}} \tag{3.21}$$

$$E\chi = Ed_{20} = 0$$

After getting d_{10} and d_{20}

$X_1$and X_2 redundant forces of members obtained by governing equation (3.5)

3.2.2 Continuity correction 2: At joint of cylindrical wall, middle ring beam, conical dome.

For solving continuity effect conical dome is treated as spherical dome whose equivalent radius is obtained by

Equivalent radius at top of cone (a_1)= $\frac{D_{1cm}}{2*Sin\alpha}$ m ...... (3.22)

Equivalent radius at bottom of cone (a_2)= $\frac{D_{2cm}}{2*Sin\alpha}$ m...... (3.23)

STIFFNESS OF CONICAL DOME

$$S_{c1}=\begin{bmatrix} \frac{E*tc}{a_1*\alpha_{1c}*(\sin\alpha)^2} & \frac{E*tc}{2\alpha_{1c}^{2}*\sin\alpha} \\ \frac{E*tc}{2\alpha_{1c}^{2}*\sin\alpha} & \frac{E*tc*a_1}{2\alpha_{1c}^{3}} \end{bmatrix} \tag{3.24}$$

Where

$$4\alpha_{1c}^{4} = \frac{12(1-\nu^2)*a_1^{2}}{tc^{2}} \tag{3.25}$$

For imaginary conical dome

For Dead Load

$$N\Phi_{Dc1} = -\frac{W_{cd}*a_1}{1+Cos\alpha} \quad \text{kg/m} \ \ldots\ldots \tag{3.26}$$

$$N\theta_{Dc1} = -W_{cd}*a_1*\left(Cos\alpha - \frac{1}{1+Cos\alpha}\right) \quad \text{kg/m}\ldots\ldots \tag{3.27}$$

$$N\Phi_{Dc1}^{0} = \frac{-W_{cd}*a_1*Sin\alpha}{(1+Cos\alpha)^2} \tag{3.28}$$

$$N\theta_{Dc1}^{0} = \frac{W_{cd}*a_1*Sin\alpha*(2+2Cos\alpha+Cos\alpha^2)}{(1+Cos\alpha)^2} \tag{3.29}$$

Where

W_{cd} is dead load of imaginary conical dome

For Water Load

From definite integral approach

$$\left[N\phi*r2*Sin^2\alpha\right]_0^{\alpha} = \int_0^{\alpha}(P_rCos\alpha - P\phi Sin\alpha)r1*r2*Sin\alpha*d\alpha \quad \ldots\ldots. \tag{3.30}$$

where

$r1=r2=a_1$

$P\Phi=0$

$$Pr = -(Y_w*H_w+Y_w*a_1(1-Cos\alpha)) = -(Y_w*a_1(\frac{H_w}{a_1}+1-Cos\alpha)) \quad \ldots. \tag{3.31}$$

and α is dummy variable. Then we get

$$N\Phi_{wc1} = -\left[\frac{-Y_w*a_1^{2}}{Sin^2\alpha}\left[\frac{H_w}{4a_1}(1-Cos2\alpha)+\frac{4Cos^3\alpha-3Cos2\alpha-1}{12}\right]\right] \quad \ldots\ldots.. \tag{3.32}$$

$$N\theta_{Wc1} = -Y_w*a_1^{2}(\frac{H_w}{a_1}+1-Cos\alpha)-N\phi_{wc1} \quad \ldots\ldots\ldots \tag{3.33}$$

$N\phi_{wc1}{}^{0} =$

$$\left[\frac{-Y_w * a_1{}^2}{Sin\alpha^4}\left[\frac{6Sin2\alpha * Sin^2\alpha - 12Cos^2\alpha Sin^3\alpha - Sin2\alpha + 1.5Sin4\alpha - 4\sin 2\alpha Cos^3\alpha}{12}\right]\right] \quad (3.34)$$

$$N\theta_{wc1}{}^{0} = -Y_w * a_1 Sin\alpha - N\phi_{wc1}{}^{0} \quad (3.35)$$

$N\Phi c_1 = N\Phi_{Dc1} + N\Phi_{wc1}$

$N\Phi c_1{}^0 = N\Phi_{Dc1}{}^0 + N\Phi_{wc1}{}^0$

$N\theta c_1 = N\theta_{Dc1} + N\theta_{wc1}$

$N\theta c_1{}^0 = N\theta_{Dc1}{}^0 + N\theta_{wc1}{}^0$

$$E\Delta H = Ed_{10} = \frac{a_1 * Sin\alpha(N\theta_{c1} - \nu * N\phi_{c1})}{tc} \quad (3.36)$$

$$E\chi = Ed_{20} = \frac{[N\theta_{c1}{}^0 - \nu N\phi_{c1}{}^0] + Cot\alpha(1+\nu)(N\theta_{c1} - N\phi_{c1})}{tc} \quad (3.37)$$

STIFFNESS OF MIDDLE RING BEAM

$$S_{mr} = \begin{bmatrix} \frac{E * A_{gm}}{R_{mm}{}^2} & 0 \\ 0 & \frac{E * I_m}{R_{mm}{}^2} \end{bmatrix} \quad (3.38)$$

For middle ring beam

$$E\Delta H = Ed_{10} = \frac{T_{mb} * R_{mm}{}^2}{A_{gm}} \quad (3.39)$$

$E\chi = Ed_{20} = 0$

After getting d_{10} and d_{20}

X_1 and X_2 redundant forces of members obtained by governing equation (3.5)

3.2.3 Continuity correction 3: at joint of conical dome, bottom Dome, Bottom ring beam.

STIFFNESS OF CONICAL DOME

$$Sc_2=\begin{bmatrix} \dfrac{E*tc}{a_2*\alpha_{2c}*(\sin\alpha)^2} & \dfrac{E*tc}{2\alpha_{2c}{}^2*\sin\alpha} \\ \\ \dfrac{E*tc}{2\alpha_{2c}{}^2*\sin\alpha} & \dfrac{E*tc*a_2}{2\alpha_{2c}{}^3} \end{bmatrix} \quad (3.40)$$

Where

$$4\alpha_{2c}{}^4=\frac{12(1-v^2)*a_2{}^2}{tc^2} \quad (3.41)$$

For Dead load

$$N\Phi_{Dc2}=-\frac{W_{cd}*a_2}{1+Cos\alpha} \quad \text{kg/m} \ \ldots\ldots\ldots \quad (3.42)$$

$$N\theta_{Dc2}=-W_{cd}*a_2*\left(Cos\alpha-\frac{1}{1+Cos\alpha}\right) \quad \text{kg/m}\ldots\ldots\ldots \quad (3.43)$$

$$N\Phi_{Dc2}{}^0=\frac{-W_{cd}*a_2*Sin\alpha}{(1+Cos\alpha)^2} \quad \ldots\ldots \quad (3.44)$$

$$N\theta_{Dc2}{}^0=\frac{-W_{cd}*a_2*Sin\alpha*(2Cos\alpha+Cos\alpha^2)}{(1+Cos\alpha)^2} \quad \ldots\ldots\ldots \quad (3.45)$$

For Water Load

From definite integral approach

$$\left[N\phi*r2*Sin^2\alpha\right]_0^\alpha=\int_0^\alpha(P_rCos\alpha-P\phi Sin\alpha)r1*r2*Sin\alpha*d\alpha$$

where

$r1=r2=a_2$

$P\Phi=0$

$$Pr=-(Y_w*(H_w+H_c)+Y_w*a_2(1-Cos\alpha))=-(Y_w*a_2(\frac{(H_w+H_c)}{a_2}+1-Cos\alpha)) \quad (3.46)$$

and α is dummy variable. Then we get

$$N\Phi_{wc2}=\left[\frac{-Y_w * a_2^2}{Sin^2\alpha}\left[\frac{(H_w+H_c)}{4a_2}(1-Cos2\alpha)+\frac{4Cos^3\alpha-3Cos2\alpha-1}{12}\right]\right] \quad (3.47)$$

$$N\theta_{Wc2}=-Y_w * a_2^2\left[\frac{(H_w+H_c)}{a_2}+1-Cos\alpha\right]-N\phi_{wc2} \quad (3.48)$$

$$N\phi_{wc2}^{0}=$$

$$\left[\frac{-Y_w * a_2^2}{Sin\alpha^4}\left[\frac{6Sin2\alpha * Sin^2\alpha-12Cos^2\alpha Sin^3\alpha-Sin2\alpha+1.5Sin4\alpha-4\sin 2\alpha Cos^3\alpha}{12}\right]\right]$$

(3.49)

$$N\theta_{wc2}^{0}=-Y_w * a_2^2 Sin\alpha-N\phi_{wc2}^{0} \quad (3.50)$$

$N\Phi_{c2}= N\Phi_{Dc2}+ N\Phi_{wc2}$

$N\Phi_{c2}^{0}= N\Phi_{Dc2}^{0}+ N\Phi_{wc2}^{0}$

$N\theta_{c2}=N\theta_{Dc2}+ N\theta_{wc2}$

$N\theta_{c2}^{0}=N\theta_{Dc2}^{0}+ N\theta_{wc2}^{0}$

$$E\Delta H=Ed_{10}=\frac{a_2 * Sin\alpha(N\theta_{c2}-\nu N\phi_{c2})}{tc} \quad (3.51)$$

$$E\chi=Ed_{20}=\frac{[N\theta_{c2}^{0}-\nu N\phi_{c2}^{0}]+\mathrm{Cot}\alpha(1+\nu)(N\theta_{c2}-N\phi_{c2})}{tc} \quad (3.52)$$

STIFFNESS OF BOTTOM RING BEAM

$$S_{br}=\begin{bmatrix}\frac{E * A_{gb}}{R_{bm}^{2}} & 0\\ 0 & \frac{E * I_b}{R_{bm}^{2}}\end{bmatrix} \quad (3.53)$$

For bottom ring beam

$$E\Delta H = Ed_{10} = \frac{(N\varphi_{bd} * Cos\theta - N_s * Cos\alpha) * R_{bm}^{\ 2}}{A_{gb}} \quad .... \quad (3.54)$$

$$E\chi = Ed_{20} = 0$$

STIFFNESS OF BOTTOM DOME

$$S_{bd} = \begin{bmatrix} \dfrac{E * t_b}{R_{m2} * \alpha_{1b} * (\sin A)^2} & \dfrac{E * t_b}{2\alpha_{1b}^{\ 2} * \sin A} \\ \\ \dfrac{E * t_b}{2\alpha_{1b}^{\ 2} * \sin A} & \dfrac{E * t_b * R_{m2}}{2\alpha_{1b}^{\ 3}} \end{bmatrix} \quad (3.55)$$

$$4\alpha_{1b}^{\ 4} = \frac{12(1-\nu^2) * R_{m2}^{\ 2}}{t_{bd}^{\ 2}}$$

For Dead Load

$$N\Phi_{Dbd} = -\frac{W_{bd} * R_{m2}}{1 + Cos\theta} \quad \text{kg/m} \ .... \quad (2.37)$$

$$N\theta_{Dbd} = -\ W_{bd} * R_{m2} * \left(Cos\theta - \frac{1}{1 + Cos\theta}\right) \quad \text{kg/m}....... \quad (2.38)$$

For Water Load:

H_D=height of water above the apex of bottom dome= (H_1+Hc-h_{bd})

$$N\Phi_{Wbd} = \frac{Yw * R_{m2}}{Sin\theta^2}\left[\left(\frac{H_D + R_{m2}}{2}\right) * (\cos\theta)^2 - \left(\frac{R_{m2}}{3} * (\cos\theta)^3\right) - \frac{H_D}{2} - \frac{R_{m2}}{6}\right] \ \text{kg/m}....(2.39)$$

$$N\theta_{Wbd} = -\ Yw * R_{m2}^{\ 2}\left[H_D + R_{m2}(1 - \cos\theta)\right] - N\phi_{Wbd} \quad \text{kg/m}..... \quad (2.40)$$

$$N\Phi_{Dbd}^{\ 0} = \frac{-W_{bd} * R_{m2} * Sin\theta}{(1 + Cos\theta)^2} \quad (3.56)$$

$$N\theta_{Dbd}^{\ 0} = \frac{W_{bd} * R_{m2} * Sin\theta * (2 + 2Cos\theta + Cos\theta^2)}{(1 + Cos\theta)^2} \quad (3.57)$$

$N\Phi_{wbd}{}^{0}=$

$$\left[\frac{Y_w * R_{m2}}{Sin\theta^4}\left(\left[\frac{-(H_D + R_{m2})}{2} * Sin2\theta\right] + \left[R_{m2} * Cos^2\theta * Sin^3\theta\right] + Sin2\theta\left[\frac{R_{m2} * Cos^3\theta}{3} + \frac{H_D}{2} + \frac{R_{m2}}{6})\right]\right)\right]$$

(3.58)

$N\theta_{wbd}{}^{0} = -Yw * R_{m2}{}^{2} * Sin\theta - N\theta_w{}^{0}$ (3.59)

Now

$N\Phi_{bd} = N\Phi_{Dbd} + N\Phi_{Wbd}$

$N\theta_{bd} = N\theta_{Dbd} + N\theta_{Wbd}$

$N\Phi_{bd}{}^{0} = N\Phi_{Dbd}{}^{0} + N\Phi_{Wbd}{}^{0}$

$N\theta_{bd}{}^{0} = N\theta_{Dbd}{}^{0} + N\theta_{Wbd}{}^{0}$

$$E\Delta H = Ed_{10} = \frac{R_{m2} * Sin\theta(N\theta_{bd} - \nu N\phi_{bd})}{t_{bd}} \quad (3.60)$$

$$E\chi = Ed_{20} = \frac{[N\theta_{bd}{}^{0} - \nu N\phi_{bd}{}^{0}] + Cot\theta(1+\nu)(N\theta_{bd} - N\phi_{bd})}{t_{bd}} \quad (3.61)$$

After getting d_{10} and d_{20}

X_1 and X_2 redundant forces of members obtained by governing equation (3.5)

3.3 EXCEL PROGRAM FOR CAPACITY OF 1000 KL

With the help of above equation a program in Microsoft excel has been developed which shows complete design of container and staging in wind zone 2_{nd}. As far as staging is concern required parameter and equation mention in chapter (4). Excel program cover the codal provision related to design of water storage tank. With the help of excel program we can predict where is section safe or not, quantity of reinforcement in container and staging as well as spacing of reinforcement.

		OBTANING DIAMETER OF INTZE TANK										
V	λ1(D2/D1)	λ2(H1/D1)	α	redian	Tanα	θ	redian	Cosθ	Sinθ	$D1^3$	D1	D2
1000	0.7	0.3	45	0.785	0.999204	44	0.767556	0.71961	0.694378	3656.13	15.41	10.78
	DEGSIGN OF CONTAINER											
	DESIGN OF CYLINDRICAL WALL											
t_w	X	D1	D_{wm}	Yw	Nθ	ction by contin	Nθ(avr)	ΔX	Tension(Kg	(Tw/1500)c	Bar on each	No of bar
0.17	0	15.40543	15.57543	1000	0	4.40E+03			Nθ(avr)*ΔX		face@cm c/	calculate
							5821.577	1.230407	7162.911	4.775274	30.76018	6.083152
	1.230407			1000	7245.748	7245.74812						
							12036.78	1.230407	14810.14	9.873428	41.01358	4.912153
	2.460815			1000	16827.81	16827.811						
							20660.64	0.984326	20336.8	13.55787	24.60815	6.745207
	3.445141			1000	24493.46	24493.4613						
							27559.72	0.787461	21702.2	14.46813	19.68652	7.198076
	4.232601			1000	30625.98	30625.9815						
							23333.15	0.689028	16077.2	10.71813	22.9676	5.332404
	4.921629			1000	35991.94	1.60E+04						
						load of column						
		DESIGN OF COLUMN				per unit height		DESIGN OF COLUMN				DESI
	Hs	Np	Nb	Hcl	Hcc	Wc/H	b_{br}	D_{br}	W_{br}	Np	Nb	Wc
	15	3	2	4.6	5	660.185	0.4	0.6	768.7046	1	0	11137.17
										2	1	24736.77
										3	2	38336.37

H1	Hcone											
4.622	2.309											
												Top ring
	actual bar	Actual Ast	σ_{st}	check	AST (Cal)	b_{tr}	b_m	t_{tr}	tw	tm	NΦ	beam(T_{tb})
ast of bar					0.006256	0.2	1	0.2	0.17	0.25	-1942.24	10905.49
0.785	8	6.28	4.029921	O.K								
2.01	6	12.06	8.011113	O.K								Ast for
												Φ= 10mm
2.01	8	16.08	10.71335	O.K								
2.01	8	16.08	11.43264	O.K								
2.01	6	12.06	8.69649	O.K								
												Height
GN OF COLUMN	WIND PRESSURE			DESIGN OF COLUMN			(stag.)			WIND	PRESS.	of
Wl. Zone	Vb(m/s)	terrain cat.	class	K1	K3	kd	Ka	ka	ght from grou	k2	member	member
iv	47	3	A	1.07	1	1	0.8	1	10	0.91	Cylin.portion	4.921629
									15	0.97	Con. Dome	2.308975
									20	1.01	Ring Beam	1.25
									30	1.06	Col.>10m	5
									50	1.12	Col<10m	10
										Nb=	1	0.6

Top ring									middle ring
beam(T_{tb})								Middle ring	beam(T_{mb})
correction	Nx	W1	W11	W2	W22	W3	W33	beam(T_{mb})	correction
2.35E+03	-3540.33961	-100	-4900.11	-3540.34	-173147	-625	-32195.659	35015.241	1.85E+04
									Ast
1.5667021									12.3553525
no of bar								Ast for	no of bar
1.386462								Φ= 16mm	6.14694153
act AST									actual steel
2.26									16.08
5.4925004									6.86849479
O.K									O.K
height from						DESIGN OF COLUMN			
ground		DESIGN OF COLUMN							total
	efec.K2	Vz	Pz kg/m2	Vz*b	H/b	cf	Ae	force(kg/m)	hprizontal.F
23.480605	1.027403024	51.66809807	160.1755	804.7529	0.319474	0.5	15.5754315	1212.76988	5968.80395
18.558975	0.998471802	50.21314694	151.2816	791.3102	0.17633	0.5	13.6318041	1012.22126	2337.1939
16.25	0.98	49.2842	145.7359	531.4711	0.115915	0.5	11.3638021	819.650748	1024.56344
15	0.97	48.7813	142.7769	28.29315	25.86207	0.6	0.58	49.686366	248.43183
10	0.91	45.7639	125.6601	26.54306		0.6	0.58	43.7297053	437.297053
						1	0.6	75.3960436	45.2376261

		DESIGN OF CONICAL DOME CONTINUE									
Dcm	α	redian	Lc	Xc	S	r_m	W4	tc	Ns(dead)	Nθc(dead)	Hw
15.9426189	45	0.785	11.268648	0	11.27	7.97131	0	0.38	-5941.82	7578.776737	4.621629464
				0.36944	10.75	7.60158	-25.56		-6230.82	7227.253081	
				0.87741	10.03	7.0932	-58.72		-6677.39	6743.908055	
				1.43156	9.243	6.5386	-92.28		-7243.76	6216.622572	
				2.00881	8.427	5.9609	-124.3		-7945.79	5667.36686	
				2.30898	8.002	5.66049	-139.8		-8367.48	5381.75389	
CG of force	force on container	c.g of	force on								
from	in cluding	force							Total		
ground	ring beam	zs	on column	PLAN	Sp	Mp	S Max	Mc	load kg	Pu/(σck*Dc^2)	Mu/(σck*Dc^2)
21.01979	10636.30758	4.897803	bottom ring								
17.7893169					14065	80501	1563	3594.38	24736.8	0.412279423	0.006656253
15.625											
			6857.28883		20967	168082	2330	5358.36	38336.4	0.638939423	0.009922881
			45.2376261		27870	290176	3097	7122.33	51936	0.865599423	0.013189508

Xc max	Nsw	$N\theta_{wc}$	Ns(DL+wL)	θ(DL+W	S	Lc	Lc-S	ΔS	Nθ(DL+LL)	Nθ(DL+LL)correct
2.3089753	-267606	52121	-273547.95	59700	11.26865	11.26865	0		59699.781	9.61E+04
	-268913	53676.61	-275143.89	60904				0.52267		
	-271044	55184.49	-277721.82	61928	10.74598	11.26865	0.52267		60903.862	60903.86244
	-273847	55996.08	-281090.64	62213				0.718671		
	-277357	55916.77	-285302.5	61584	10.02731	11.26865	1.24134		61928.402	61928.4023
	-279449	55502.62	-287816.28	60884				0.784004		
					9.243303	11.26865	2.025345		62212.705	62212.70522
								0.816671		
					8.426632	11.26865	2.842016		61584.138	61584.13784
								0.424669		
					8.001963	11.26865	3.266685		60884.377	1.24E+05
	USE									
	OF									
	SP16									
d'/Dc	p/fck	p				dia of tie		required		
			minimum	ΦL	no	bar	Svc			
0.075	0.1425	0.036	steel	20	of bar	Φt	43.86518	Svc	Ast(m^3)	θc
			10067.625	314	34	8	75	43.87	3.4623421	30.42049965
							100			
							25			

Nθ(avr)	Nθ(avr)*ΔS	Ast(cm^2)	no of bar	Bar on each	actual no	σ_{st}			
			calculated	face= @cm c/	of bar used			ast(cm^2	Ast(cm^2)
78518.73334	41039.35643	21.59966128	10.74610014	8.71116032	12	10.01595	O.K	2.01	24.12
61416.13237	44137.97644	23.23051392	11.55746961	11.9778454	12	10.77219	O.K	2.01	24.12
62070.55376	48663.58902	25.61241528	12.74249516	11.2000633	14	11.73474	O.K	2.01	28.14
61898.42153	50550.66311	26.60561217	13.23662297	11.6667326	14	12.18979	O.K	2.01	28.14
92862.30074	39435.74647	20.75565604	10.32619703	7.07781776	12	9.624579	O.K	2.01	24.12
	moment	DESIGN OF BRACING			twisting				
	at				momentat	DESIGN OF BRACING			
	brace			Φ		brace			
θc(redian)	m(max)	Ast	Ast(min)	16	no of bar	T(Tθ)br	Vθbr	Lbr	Vubr
0.530937879	12625.75731	637.4978832	491.5662651	200.96	4	631.2879	2525.151462	2.053988	8454.815
	17601.12571	907.2265761	491.5662651		5	880.0563	3520.225143		11786.56

		bottom								
r Avg	AST(CAL)	Domeang(θ	redian	W(kg/m^2)	D2	R_{m2}	WR	NΦ(dead)	Nθ(dead)	H_D
		0	0	350	10.78380208	7.835077467	-2742.277113	-1371.138557	-1371.14	4.773844348
7.786444	0.54815	5	0.087222					-1373.749675	-1358.1	4.773844348
		10	0.174444					-1381.622924	-1319.03	4.773844348
7.347389		15	0.261667					-1394.879285	-1254.05	4.773844348
		20	0.348889					-1413.724804	-1163.34	4.773844348
6.815902		25	0.436111					-1438.457468	-1047.15	4.773844348
		30	0.523333					-1469.477383	-905.768	4.773844348
6.249752		35	0.610556					-1507.300857	-739.528	4.773844348
		40	0.697778					-1552.579263	-548.751	4.773844348
5.810698		44	0.767556					-1594.708577	-378.662	4.773844348
	DESIGN OF BRACING									
	CODAL VALUE									
			%pt	ζe		ζe	ζv	check	Sv	b1
Ve	%pt	lower limit	upper limit	lower limit	upper limit					
16469.95	0.277173	0.25	0.5	0.36	0.49	0.374129956	0.716084743	O.k	288.9316	0.35
22960.18	0.394446	0.25	0.5	0.36	0.49	0.435112095	0.99826864	O.k	175.4424	

									CONTINUITY CORRECTION CONTINU		
$Yw*R_{2m}$	NΦ(water)	Nθ(water)	NΦ(DL+WL	Nθ(DL+WL	σ(Max)	Ast=	No of bar	Rm(dome)	t	v	α
7835.077	-18701.7	-18701.72014	-20072.9	-20072.9	13.38191	.24%bd	both	11.13297	0.08	0.15	15.43725
7835.077	-18760	-18876.78085	-20133.8	-20234.9	13.48992	360 mm^2	direction	7.835077	0.14	0.15	9.789649
7835.077	-18934	-19401.07783	-20315.7	-20720.1	13.81341		8#8Φ @17c	wall			
7835.077	-19221.1	-20271.97435	-20616	-21526	14.35068			7.787716	0.17	0.15	8.85707
7835.077	-19616.8	-21485.13865	-21030.5	-22648.5	15.09898			7.787716	0.17	0.15	8.85707
7835.077	-20114.7	-23034.63989	-21553.1	-24081.8	16.05452			Conical wall			
7835.077	-20706.7	-24913.08612	-22176.2	-25818.9	17.21257			11.27763	0.38	0.15	7.128966
7835.077	-21382.7	-27111.80822	-22890	-27851.3	18.56756			7.628338	0.38	0.15	5.863172
7835.077	-22130.5	-29621.0959	-23683.1	-30169.8	20.11323						
7835.077	-22771	-31845.12344	-24365.7	-32223.8	21.48252			top ring beam			
								7.802716	0.2		
								middle ring beam			
								8.202716	0.25		
								bottom ring beam			
								5.681901	1.25		
d1	X1	Y1	(X1+y1)/4	SV_{br}	Vc	Vs					
0.575	358.4721	583.472136	470.9721	288.9	0.492957	0.063903					
							Vc stag	Vsstag			
				175.4		0.081651					
							119.9	4.252			

E

A or θ	COSA	COS2A	SINA	SIN2A	COT A	f_{ck}	E=Kg/m^2)	S11	S12	S22		W(dome)
44	0.71961	0.035678	0.694378	0.999363	1.036337	25	2.5E+09	2413553	604315.7	302622.3	top dome	300
44	0.71961	0.035678	0.694378	0.999363	1.036337			9463796	2629709	1461436	ottom dom	350
								6161533	-2708811	2381764		
								6161533	-2708811	2381764		
45	0.707388	0.000796	0.706825	1	1.000797			23651320	13222955	14785352	onical.dom	950
45	0.707388	0.000796	0.706825	1	1.000797			42514495	19548628	17977344		950
								1642512	0	5475.038	p ring beam	
								9288909	0	48379.73	dle ring beam	
								56142363	0	7310203	om ring beam	

CONTINUITY EFFECT CONTINUE

$N\Phi_d$	$N\theta_d$	$N\Phi_d^0$	$N\theta_d^0$	Yw	D,HW,Hw+I	$N\Phi_w$	$N\Phi w^0$	$N\theta_w$	$N\theta_w^0$	Nφfin	$N\Phi f^0$	Nθfin
-1942.24	-461.182	-784.275	3103.42216									
-1594.71	-378.662	-643.943	2548.12065	1000	4.773844	-22771	-9403.84	-31845.1	-33223	-23149.7	-10047.8	-32223.8
-6274.93	-1303.85	-2597.7	10170.4421	1000	4.621629	-34832.9	65238.5	-54503.8	-155136	-36136.8	62640.8	-55807.7
-4244.45	-881.939	-1757.12	6879.42401	1000	6.930605	-30448.2	29848.91	-39448.3	-70980.1	-31330.1	28091.79	-40330.3

CONTINUITY EFFECT CONTINUE

Nθf^0	EΔH	Ed10	EX	E*d20	s11*d10	S12*d20	s21*d10	S22*d20	FER1	FER2	cylindrical wall	Kw/E*
	-16412.5	-16412.5	62327.08	-62327.1	-4E+10	-3.8E+10	-9.9E+09	-1.9E+10	30.91105	11.51195		
-30674.8	-1117299	-1117299	-285586	285586.3	-1.1E+13	7.51E+11	-2.9E+12	4.17E+11	3929.152	1008.322		
												0.000419
												0.000419
-144965	-1056978	-1056978	-465793	465792.5	-2.5E+13	6.16E+12	-1.4E+13	6.89E+12	7535.909	2835.787		
-64100.7	-505572	-505572	-207034	207033.9	-2.1E+13	4.05E+12	-9.9E+12	3.72E+12	6978.767	2464.529		
	16598807	16598807	0	0	2.73E+13	0	0	0	-10905.5	0		
	9423938	9423938	0	0	8.75E+13	0	0	0	-35015.2	0		
	9291165	9291165	0	0	5.22E+14	0	0	0	-208651	0		

CONTINUITY EFFECT CONTINUE

horizontal thrust

H	EΔH	E*d10	Ex	Ed20	s11 d10	s12 d20	s21 d10	s22 d20	FER1	FER2	joint 1	∑FER1
												-12272.2
1397.653452	1134177	1134177	-1289914	1289914	6.99E+12	-3.5E+12	-3.1E+12	3.07E+12	-1397.65	0	joint 2	-63471.3
35991.93667	29206968	29206968	-3.3E+07	33217464	1.8E+14	-9E+13	-7.9E+13	7.91E+13	-35991.9	-6.3E-12		
											joint 3	-197743

CONTINUTY CORRECTION OVER

∑FER2	∑s11	∑s12	∑s22	U1	U2	X1(kg/m)	∑X1	X2	∑X2	
11.51195	10217597	-2104496	2689862	1.43E-03	1.12E-03	4.16E+03	0.00E+00	1.21E+03	-5.37E-13	member
				1.43E-03	1.12E-03	-8.56E+03		-1.22E+03		Top dome
				1.43E-03	1.12E-03	4.40E+03		6.11E+00		
2835.787	39101761	10514144	17215496	2.00E-03	-1.38E-03	-2.00E+04	0.00E+00	-8.70E+03	-1.92E-12	top ring beam
				2.00E-03	-1.38E-03	-1.65E+04		8.77E+03		
				2.00E-03	-1.38E-03	3.64E+04		-6.69E+01		cylindrical wal
3472.851	1.08E+08	22178337	26748984	2.24E-03	-1.98E-03	6.33E+04	2.91E-11	1.05E+04	-5.91E-12	
				2.24E-03	-1.98E-03	-8.31E+04		-1.45E+04		Middle ring bea
				2.24E-03	-1.98E-03	1.99E+04		3.99E+03		
										conical wall
										conical wall
										bottom dome
										bottome ring bea

TO FIND VOLUME OF CONCRET AND VOLUME OF REINFORCEMENT

D1	Angle A	redian	htd	t(m)	b	Area	I moment	t secα	D1+tsecα	Rm1	Hw or Hc
15.4054315	44	0.767556	3.081086	0.08						11.13297	
15.4054315				0.2	0.2	0.04	0.000133			7.802716	
15.4054315				0.17						7.787716	4.671629
15.4054315				0.25	1	0.25	0.001302			8.202716	
15.4054315				0.38				0.537187	15.94262	7.971309	2.308975
10.7838021				0.38				0.537187	11.32099	5.660495	
10.7838021	44	0.767556	2.15676	0.14						7.835077	
10.7838021				1.25	0.58	0.725	0.094401			5.681901	

						FOR	STAGGING CONTINUE					
						BOTTOM					ALL DIMENSION OF L	
			UDLon bottom			RING BEAM		θ_{bt} for				
Vc(m^3)	ΣVc	Vs	ΣVs	ring beam	AST_4	no of colun	α_b(angle)	(T max)	R_{m2}	D_c(dia.column)	b_b	t_b
4.380383	97.23	0.02103	1.165	198656.5		18	0.174444	0.073763	5.681901	0.6	0.58	1.25
1.960042		0.011074										
38.8407		0.399179										
12.87826		0.082833			1.256913							
35.39458		0.633097										
3.776422		0.018127										

.ENGTH IN METER

$\theta_0<\alpha_0$(at su	$\theta_0<\theta_b<\alpha_0$	W_{drb}	W_{ud}(kg/m)	Vu(shear)	M(mθ)	T(Tθ)b	Vθ	Mu	Tu	Ve	Mt	Me1
0.052799	0.052799	1812.5	200469	138559.4	-15127.5	-2057.08	-5674.71	-22691.25	-3085.62	216351.1	-5726.87	-28418.1
	0.073763			114680.7	-7.2E-10	-2210.68	-6098.44	-1.08E-09	-3316.02	181168.7	-2855.68	-2855.68
	0.104667			79480.02	17111.86	-1930.33	-5325.06	25667.786	-2895.5	127207.6	-1703.23	23964.55
	0.122111			59610.02	24035.69	-1568.56	-4327.07	36053.531	-2352.84	95905.63	-1384.03	34669.5
	0.139556			39740.01	28982.79	-1103.25	-3043.45	43474.178	-1654.87	64175.18	-973.455	42500.72
	0.174444			0	32941.37	0	0	49412.05	0	0	0	49412.05

				ΦL in mm								
				25					CODAL VALUE			Ast req.
Mt-Mu	Me2	Ast(Me1)	Astmin	Ast of bar	no of bar	Ast(Me2)	Φ=20mm	no of bar	c max(MPA	%pt	ζc	Ast_{rbL}
-16964.4	0	652.4741	1484.94	490.625	4	0	314	0	3.1	0.15	0.29	1484.94
2855.682	2855.682	64.66408	1484.94	490.625	4	1484.94		5		0.25	0.36	1484.94
-23964.6	0	548.8721	1484.94	490.625	4	0		0		0.5	0.49	1484.94
-34669.5	0	798.7775	1484.94	490.625	4	0		0		0.75	0.57	1484.94
-42500.7	0	983.5332	1484.94	490.625	4	0		0		1	0.64	1484.94
-49412.1	0	1147.987	1484.94	490.625	4	0		0		1.25	0.7	1484.94
										1.5	0.74	
										1.75	0.78	
										2	0.82	
										2.25	0.85	
										2.5	0.88	
										2.75	0.9	
										3	0.92	

		FROM FACE OF COLUMN TO MAXIMUM TORSION i.e θb =						0	to	0.05279923	
%pt cal	trail for pt%				for ζc						
0.426707	lower limit	upper limit	lower limit	upper limit	lower limit	upper limit	lower limit	upper limit	ζe	ζve	CHEK
0.426707	0.15	0	0.25	0.5	0.29	0	0.36	0.49	0.451888	3.045054226	O.k
0.426707	0.25	0			0.36	0					
0.426707	0	0.5			0	0.49					
0.426707	0	0.75			0	0.57					
0.426707	0	1			0	0.64					
	0	1.25			0	0.7					
	0	1.5			0	0.74					
	0	1.75			0	0.78					
	0	2			0	0.82					
	0	2.25			0	0.85					
	0	2.5			0	0.88					
	0	2.75			0	0.9					
	0	3			0	0.92					
			FROM θb =		0.104667	to	0.12211111				
			trail for pt%				for ζc				
			lower limit	upper limit	lower limit	upper limit	lower limit	upper limit	lower limit	upper limit	ζe
			0.15	0	0.25	0.75	0.29	0	0.36	0.57	0.434216867
			0.25	0			0.36	0			
			0	0.5			0.	0.49			

										FROM θ_b =	0.052799	to
									FOR %pt			
Asv	b1	d1	Sv_{br}	X1	Y1	X1+Y1/4	Sv_{br}		lower limit	upper limit	lower limit	upper limit
226	0.53	1.2	54.252	539.4721	1209.472	437.2361	54.2522		0.15	0	0.25	0.5
									0.25	0		
									0	0.5		
									0	0.75		
									0	1		
									0	1.25		
									0	1.5		
									0	1.75		
									0	2		
									0	2.25		
									0	2.5		
									0	2.75		
									0	3		
								FROM θb :	0.122111	to	0.139556	
						trail for pt%				for ζc		
ζve	CHEK	Sv	correct			lower limit	upper limit	lower limit	upper limit	lower limit	upper limit	lower limit
1.349833	O.k	153.6507	Sv_{br}			0.15	0	0.25	0.5	0.29	0	0.36
			153.6507			0.25	0			0.36	0	
						0	0.5			0	0.49	

0.073763										FROM θb :	0.073762974
for ζc								trail for pt%			
lower limit	upper limit	lower limit	upper limit	ζe	ζve	CHEK	Sv_{br}	lower limit	upper limit	lower limit	upper limit
0.29	0	0.36	0.49	0.451888	2.55	O.k	67.05707	0.15	0	0.25	0.5
0.36	0						But	0.25	0		
0	0.49						67.06	0	0.5		
0	0.57							0	0.75		
0	0.64							0	1		
0	0.7							0	1.25		
0	0.74							0	1.5		
0	0.78							0	1.75		
0	0.82							0	2		
0	0.85							0	2.25		
0	0.88							0	2.5		
0	0.9										
0	0.92										
										FROM θb :	0.139555556
								trail for pt%			
upper limit	ζe	ζve	CHEK	Sv_{br}				lower limit	upper limit	lower limit	upper limit
0.49	0.451888	0.90324	O.k	311.6967				0.15	0	0.25	0.5
								0.25	0		
				BUT				0	0.5		

to	0.104667										Side
for ζc									longitudinal		reinforcemer
lower limit	upper limit	lower limit	upper limit	ζe	ζve	CHEK	Sv_{br}	Vc	Vs_{brL1}	Vs_{brL2}	Vs_{brL3}
0.29	0	0.36	0.49	0.45188755	1.790395724	O.k	105.1058	25.8697	0.058875	0	0.02587
0.36	0						but		0.023376	0.018701	
0	0.49						105.1		0.03446	0	
0	0.57								0.019452	0	
0	0.64								0.019452	0	
0	0.7								0.038904	0	
0	0.74										
0	0.78										
0	0.82										
0	0.85										
0	0.88										
to	0.174444										
for ζc											
lower limit	upper limit	lower limit	upper limit	ζe	ζve	CHEK	Sv_{br}		FINAL Vs		
0.29	0	0.36	0.49	0.451888	0	O.k	no need		0.478178		
0.36	0										
0	0.49						because				

CHAPTER 4

ANALYSIS AND DESIGN OF STAG ING

4.1 INTRODUCTION

The design of container is described in chapter (2). A framework of vertical circular columns connected by horizontal braces of rectangular cross section having ring beam at top has been proposed for the staging. The analysis for horizontal forces has been carried out by the tube analogy method and the design is based on limit state method.

4.2 BOTTOM RING BEAM

The bottom ring beam is considered as a part of the staging because its design is governed by the number of columns in the staging. The bottom ring beam is subjected to the total vertical load of the container and the water including self weight in the vertical direction. In addition it is also subjected to a radial force which is the difference of the horizontal components of the meridional thrust from the conical dome and bottom dome.

The bottom ring beam is analyzed as a beam curved in plan and continuous over column supports. In this case, the center of gravity of load lie out side the line joining it supports. This cause an over turning of the beam which can be prevented if bam fixed at its ends or continuous over the supports. The design is based on the recommendation of limit state methods per IS: 456-2000.the number of column is fixed by angle α_b so that the edge beam is connected to the column directly.

4.2.1 Design Forces:

Bending moment in bottom ring beam at any point

$M_{\theta b} = W_{rb} * R_{bm}^{2}(\alpha_b \sin\theta_b + \alpha_b \cot\alpha_b \cos\theta_b - 1)$ kg m ..... (4.1)

Torsion moment in bottom ring beam at any angle θ_b

$T_{\theta b} = W_{rb} * R_{bm}^{2}(\alpha_b - \theta_b - \alpha_b \cos\theta_b + \alpha_b \cot\alpha_b \sin s\theta_b)$ kg m .... (4.2)

Where W_{rb} is load per unit length in bottom ring beam which is obtained by governing equation

$W_{rb} = N\phi_{bd} \sin\theta + Ns \sin\alpha$ kg/m .... (4.3)

Hoop compression in bottom ring beam

$T_{bb} = N\phi_{bd} Cos\theta + NsCos\alpha$ kg/m (4.4)

θ_b =angle (radian) at the center that a section make from left support

α_b =semi central angle subtended by supports at the center.

Angle between the center of column support and the point of maximum torsion

$$\theta_{bt} = \alpha_b - Cos^{-1}\left(\frac{\sin\alpha_b}{\alpha_b}\right) \quad .... \tag{4.5}$$

(a) longitudinal reinforcement

The code IS 456:2000[17] recommended a simplified skew- bending based formulation for design of longitudinal reinforcement to resist torsion combined with flexure in beam with rectangular cross section. The torsion moment Tu is converted into an effective bending moment M_t defend as follow

$$Mt = \frac{T_u(1+\frac{t_{rb}}{b_{rb}})}{1.7} \text{ kg m} \tag{4.6}$$

So calculated M_t, is combined with actual bending moment M_u at the section, to give equivalent bending moment Me_1 and M_{e2}

$Me_1 = M_t + M_u$.(4.7)

$Me_2 = M_t - M_u$ (4.8)

The longitudinal reinforcement area Ast_{rbL1} is design to resist the equivalent moment Me_1,and reinforcement is to be located in the flexural tension zone .In addition if $Me_2 > 0$, then reinforcement area Ast_{rbL2} is to be designed to resist this equivalent moment, and this reinforcement is to be located in the flexural compression zone. It follow from the above that in the limiting case of pure tension i.e M_u=0 equal to longitudinal reinforcement is required at top and bottom of rectangular beam, each capable of resisting an equivalent bending moment equal to M_t.

The longitudinal reinforcement should not less than 0.85 $b_{rb}t_{rb}/fy$

(b) Transverse reinforcement

Vertical shear force at any angle θ_b

$$V\theta_{rb} = W_{rb} * R_{bm}(\alpha_b - \theta_b) \quad \text{kgm} \tag{4.9}$$

Equivalent shear

$$\tau_{ve} = \frac{Vu + 1.6\frac{T_u}{b_{rb}}}{b_{rb}t_{rb}} \quad \text{N/mm}^2 \ \ldots. \tag{4.10}$$

If τ_{ve} exceed τ_c, max the section has to be suitably redesign by in creasing the cross sectional area (especially width) and /or improving grade of concrete

Spacing of Shear reinforcement at support section

(i)when $0.5\tau_c < \tau_{ve} < \tau_c$

$$S_{vrv} = \frac{0.87fy * A_{Sv}}{0.4b_{rb}} \ \ldots.\text{mm} \tag{4.11}$$

(ii) when $\tau_c < \tau_{ve} < \tau_c$ max

$$S_{vrb} = \frac{0.87fy * A_{Sv}}{(\tau_{ve} - \tau_c)b_{rb}} \ \ldots..\text{mm} \tag{4.12}$$

Where τ_c is obtained from Table 19 of IS: 456

The shear reinforcement at the point of maximum torsion

$$A_{sv} = \frac{Tu_{rb}}{b_1d_1(0.87f_y)} + \frac{Vu_{rb} * Sv_{rb}}{2.5d_1(0.87f_y)} \quad \text{mm}^2 \ldots\ldots\ldots.. \tag{4.13}$$

But total transverse reinforcement shall not be less than $\frac{(\tau_{ve} - \tau_c)b_{rb} * Sv_{rb}}{0.87fy}$

Where A_{sv} is the total area of two legs of the stirrup's is the center to center to spacing of stirrup; b1 and d1 are the center to center distance between the corner bars along the width and depth respectively; and T_u and V_u are the factored twisting moment and factored shear force acting at the section of consideration.

Specific maximum limits to spacing S_v of the stirrups provide as tensional reinforcement, to control crack widths and to control the fall in tensional stiffness on account of tensional cracks, should not exceed x_1 or $(x_1+y_1)/4$ or 300mm where x_1 and y_1 are respectively ,the short and long center to center dimensions of the rectangular closed stirrup.

4.3 COLUMNS

The columns are to be designed for vertical loads due to the weight of the container, the weight of water and the self weight as well as horizontal forces due to wind. Generally the columns provided have the same cross-sectional area and are placed symmetrically. Therefore the vertical loads are shared equally by all the columns.

The lateral forces induce bending moments, shear forces and axial forces in the columns. The magnitude of these reactions depends upon the condition of the end-fixidity of the columns at the top and bottom. This problem is statically indeterminate and involves the analysis of the tower as a space frame.

The analysis for horizontal loads is done by the tube analogy method. In this method the tower is assumed to behave as a cantilever under the action of horizontal forces with the neutral axis passing through the bending axis of the tower. The columns which are built monolithically with the container base at the top, braces in the middle and the foundation at the bottom are considered infinitely rigid and therefore, the rotation of columns at these points is considered to be zero. If the bending moment and shear forces due to the lateral loads on the tower are calculated on this equivalent vertical cantilever beam at the horizontal sections passing through points of inflexion, then the bending stresses in the equivalent vertical cantilever beam will give the vertical forces in the columns and the shear stresses in the cantilever beam will give the horizontal shear force in the columns at their points of inflexion. The detailed formulation has been explained by Jai Krishna and Jain[11]

4.3.1 Dimensions:

The columns are normally designed as short columns and the minimum number of panels in the staging is given by

$$Np=\frac{H_s}{12 * D_c} \quad ..... \qquad (4.14)$$

Np is rounded off to the nearest higher number to get whole number

There for the number of braces is given by

$$Nb=Np-1 \qquad (4.15)$$

and clear panel height is obtained from

$$H_{cL}=Hs-Nb*t_{br}. \quad m..... \qquad (4.16)$$

The center to center panel height of the column is given by

$$H_{cc}=H_{cL}+t_{br} \quad m....... \qquad (4.17)$$

The effective length of the brace is obtained from

$$L_{be}=\frac{(D_2+D_c)*\pi}{Nc}-D_c \quad m...... \qquad (4.18)$$

And clear length of brace

$$Lb_{cL}=L_{be}-D_c \quad m...... \qquad (4.19)$$

4.3.2 Wind Loads:

The basic wind pressure is taken as per provision of IS: 875(Part-3)-1987[20] for place where the structure is suited. The wind load on the container and staging is computed as follow:

The wind force on container member is given by

$$W_{f\,cylinder}=cf_{cylinder}*Pd_{cylinder}*Ae_{cylinder} \quad kg..... \qquad (4.20)$$

$$W_{f\,cone}=cf_{cone}*Pd_{cone}*Ae_{cone} \quad kg..... \qquad (4.21)$$

$$W_{f\,bottom\,ring\,beam}=cf_{bottom\,ring\,beam}*Pd_{bottom\,ring\,beam}*Ae_{bottom\,ring\,beam} \quad kg... \qquad (4.23)$$

Resultant force on container

$$Wf_{cn}= W_{f\,cylinder}*H_{cw}+ W_{f\,cone}*H_c+ W_{f\,bottom\,ring\,beam}* t_{rb} \quad kg..... \qquad (4.24)$$

This force acting at the Zs distance from bottom dome portion

The Wind force on column of each panel is

$Wf_c = cf_c * Pd * (0.5Nc+1) H_{cL} * Dc$ kg ..... (4.25)

Wind load on braces of each panel are

$W_{fbr} = cf_{br} * Pd * (D_2 + 2D_c)$ Kg...... (4.26)

There for the maximum panel shear at the mid height of first panel from ground level is

$Sp = Wf_{cn} + (Np-0.5) Wf_c + (Np-1) Wf_{br}$ Kg...... (4.27)

and the panel moment at the mid height of the above panel

Mp=

$$\left[W_{fcn}\{Zs + (Np-1)H_{cc} + 0.5H_{cL}\} + 0.5Wfc\{0.25H_{cl} + (Np-1)H_{cc}\} + \{(Wf_c + Wf_{br})(0.5Np^2 \_ Np + 0.5$$

Kgm.......(4.28)

4.3.3 Forces on Columns:

(a) The gravity forces on each column of the lowest panel is given by

$$Wc = \frac{W_{rb}}{Nc} + 2500(\mathrm{d_{br}} * \mathrm{b_{br}} + \Pi \mathrm{D_c}^2 * \mathrm{H_{cL}}) * \mathrm{N}b \qquad \text{Kgm......} \qquad (4.29)$$

(b)Horizontal forces:

Maximum thrust on leeward column of the lowest panel lying farthest from the bending axis

$$\text{Tc} = \frac{4M_p}{N_c * (D_2 + D_c)} \qquad \text{Kg......} \qquad (4.30)$$

Maximum bending moment on the column of lowest lying on the bending axis

$$\text{Mc} = \frac{\mathrm{H_{cL}} * \mathrm{Sp}}{\mathrm{Nc}} \qquad \text{Kgm......} \qquad (4.31)$$

4.3.4 Reinforcement in columns

(a) Longitudinal Reinforcement

Each column of the staging is to be designed for the maximum thrust and the combined effect of thrust and moment .The reinforcement is calculated by using design chart (for uniaxial eccentric compression)in SP:16 .In circular column at least six bars are required and they are equally placed

(b) Transverse Reinforcement:

The pitch and diameter of the lateral ties or helical reinforcement is kept as per clause 26.5.3.2 of IS: 456_2000 [17]. The minimum diameter should be 6mm.

4.4 BRACES

Bending moment in a brace is developed at the junction due to shear force acting at the mid height of columns. The bending moment has to be resisted by the two braces meeting at the junction. The bending moment in the brace about the vertical axis of its section is almost zero and even the twisting moment is negligible. Thus the brace bear the joint moment mostly by developing bending moment about the horizontal axis of its section. These moments in the brace can easily be calculated by considering the statically equilibrium of moments at the joint.

4.4.1 Wind Forces on Braces:

Moment in the lowest brace rigidly connected to the column making an angle θc with the bending axis of tower is given by

$$M_{br} = \frac{2Sp * H_{cc} * Cos^2\theta c * Sin(\theta c + \alpha_b)}{N_c * Sin2\alpha_b} \quad \text{Kgm......} \qquad .(4.32)$$

For maximum moment in the brace

$$Tan\theta_c = \frac{-3Tan\alpha_b + \sqrt{((3Tan\alpha_b)^2 + 8)}}{4} \qquad (4.33)$$

Shear force in the lowest brace lying between the column making angles $(\theta c - \alpha_b)$ and θc with the bending axis of the tower.

$$V_{br} = \frac{2S_p * H_{cc}}{Nc * Sin2\alpha_b}\left\{Cos^2\theta_c Sin(\theta_c + \alpha_b) - Cos^2(\theta_c - 2\alpha_b)Sin(\theta_c - 3\alpha_b)\right\} \quad \text{kg....(4.34)}$$

For maximum values of shear force $\theta_c = \alpha_b$ I.e. wind blowing parallel to the brace. Hence maximum shear force

$$V_{br}(Max) = \frac{4S_p * H_{cc} * Cos^2\theta_c}{Nc} \quad \text{kg...} \quad (4.35)$$

Twisting moment (Tu_{br})=5% M_{br}(Max)

Equivalent shear

$$\tau_{ve} = \frac{Vu_{br} + 1.6\frac{T_{ubr}}{b_{br}}}{b_{br}t_{br}} \quad \text{N/mm}^2\text{....} \quad (4.36)$$

If τ_{ve} exceed $\tau c max$ the section has to be suitably redesign by in creasing the cross sectional area (especially width) and /or improving grade of concrete

4.4.2 Reinforcement in bracing

(a)The longitudinal reinforcement area Ast_{brL} is design to resist moment M_{br} and reinforcement is to be located in the flexural tension zone. The moment in braces may be positive or negative according to the direction of wind .hence the brace must be reinforced equally at top and bottom. The longitudinal reinforcement should not less than 0.85 $b_{br} t_{br}/fy$

(b)Spacing of Shear reinforcement at support section

(i) When $0.5\tau c < \tau_{ve} < \tau_c$

$$S_{vbr} = \frac{0.87 fy * A_{Sv}}{0.4 b_{br}} \quad \text{mm.......} \quad (4.37)$$

(ii) when $\tau_c < \tau_{ve} < \tau_{c\ max}$

$$Sv_{br} = \frac{0.87 fy * A_{Sv}}{(\tau_{ve} - \tau_c) b_{br}} \quad \text{mm.......} \quad (4.38)$$

Where τ_c is obtained from Table 19 of IS: 456-2000[17]

The shear reinforcement at the point of maximum torsion

$$A_{sv} = \frac{Tu_{br}}{b_1 d_1 (0.87 f_y)} + \frac{Vu_{br} * Sv}{2.5 d_1 (0.87 f_y)} \quad mm^2 \ldots\ldots \qquad (4.39)$$

But total transverse reinforcement shall not be less than $\frac{(\tau_{ve} - \tau c) b_{br} * S_{v\,br}}{0.87 fy}$

Where Asv, is the total area of two legs of the stirrups is the center to center to spacing of stirrup; b1 and d1 are the center to center distance between the corner bars along the width and depth respectively; and T_u and V_u are the factored twisting moment and factored shear force acting at the section of consideration.

Specific maximum limits to spacing S_v of the stirrups provide as tensional reinforcement, to control crack widths and to control the fall in tensional stiffness on account of tensional cracks, should not exceed x_1 or $(x_1+y_1)/4$ or 300mm where x_1 and y_1 are respectively ,the short and long center to center dimensions of the rectangular closed stirrup.

4.5 RELEVENT PARAMETER

4.5.1 Volume of concrete

$$V_{coL} = \frac{\Pi}{4} D_c^2 Hs * Nc \quad m^3 \ldots\ldots \qquad (4.40)$$

$$V_{br} = Nc * Nb * Lbr * Bbr * dbr \quad m^3 \ldots\ldots \qquad (4.41)$$

$$V_{rb} = \Pi * D_{bmr} * t_{br} * brb \quad m^3 \ldots\ldots \qquad (4.42)$$

$$Vc_{staging} = V_{coL} + V_{br} + Vrb \quad m^3 \ldots\ldots \qquad (4.4.3)$$

4.5.2 Volume of steel

(a)Bottom ring beam

$$Vs_{rbL} = \Pi D_{bmr} (Ast_{rbL1} + Ast_{rbL2}) \quad m^3 \ldots \qquad (4.44)$$

$$Vs_{rbt} = Nc(b_{rb} + t_{rb} - 0.25) * Asv \sum \theta_i * D_{2bmr} \quad m^3 \ldots \qquad (4.45)$$

$$Vs_{rb} = Vs_{rbL} + Vs_{rbt} \quad m^3 \ldots \qquad (4.46)$$

(b)Column

$$Vs_{cL} = Nc * Hs * Asc \quad m^3 \ldots \qquad (4.47)$$

$$Vs_{ct} = \frac{\Pi d_{trv}^2}{4} \left(\frac{Hs}{S_{trv}}\right) * \Pi * D_c \quad m^3 \ldots \qquad (4.48)$$

$Vsc = Vs_{cL} + Vs_{ct}$ m^3.... (4.49)

(c)Braces

$Vs_{brL} = L_{br} * Ast_{br} * Nc * Nb * 2$ m^3.... (4.50)

$$Vs_{brt} = \frac{L_{br}}{Sv_{br}} * 2(b_{br} + t_{br} - 0.25) * Asv_{br} \quad m^3.... \quad (4.51)$$

$Vs_{br} = Vs_{brL} + Vs_{brt}$ m^3.... (4.52)

$Vst = Vs_{rb} + Vsc + Vs_{rb}$ m^3.... (4.53)

4.5.3 Cost of staging

On the basis of the present prevailing rates of material the rates of various items are given as under:

(3) Cement concrete of grade M25 in superstructure excluding cost of centering and shuttering — Rs.2900 per cu.m

(4) Tor steel reinforcement including cutting,bending,placement and its wastage — Rs18000 per ton

There for the cost of staging $= Vc_{staging} * 29000 + 78.5 * 18000 * Vst$ Rupees ...... (4.54)

CHAPTER 5

RESULTS

5.1 INTRODUCTION

The alternative design of container and staging are obtained by using Excel program which is explain in chapter (3). The effect of various parameters on the cost of container is studied and design for optimal parameter is given in the form of graph and table.

5.2 CONTAINER

The effect of following parameters is studies by varying the parameter as per table-1.1

(1)Staging-container diameter ratio λ_1 (D_2/D_1)

(2)Height diameter ratio, λ_2 (H_1/D_1)

(3)Horizontal angle of cone α

The result are tabulated in tables 5.1 to5.3 and graph plotted between cost of container and above parameter are drawn for all six different capacities are shown in figure 5.1 to 5.3

Table 5.1 Quantity of concrete, reinforcement and cost of container for different value of λ_1 and $\lambda 2=0.3$ $\alpha=50^0$

Capacity KL	λ_1	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container in lac	Capacity KL	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container in lac
300	0.5	30.46	0.30671	0.4379844	1000	133.9	1.2241	1.783845
	0.55	30.51	0.30499	0.4361554		131	1.25376	1.809267
	0.6	30.59	0.30438	0.4356982		127.9	1.25323	1.7995933
	0.65	30.72	0.30053	0.4316843		124.5	1.24948	1.7855015
	0.7	30.91	0.32293	0.4577693		120.9	1.20677	1.726183
	0.75	31.18	0.33468	0.4719532		116.9	1.18731	1.6924585
500	0.5	51.48	0.48885	0.7065876	1250	194.9	1.76754	2.5803188
	0.55	50.97	0.50561	0.7241983		190.1	1.71954	2.511553
	0.6	50.43	0.50963	0.727228		184.8	1.70891	2.4841874
	0.65	49.89	0.52192	0.7396584		179.1	1.70429	2.4623788
	0.7	49.33	0.52125	0.7372871		172.9	1.6667	2.4015303
	0.75	48.78	0.49681	0.7078137		166.2	1.65495	2.3684856
750	0.5	87.74	0.86091	1.2358695	1500	259.6	2.30934	3.385619
	0.55	86.26	0.85358	1.2232238		252.5	2.31486	3.3710966
	0.6	84.67	0.84095	1.2042191		244.7	2.26603	3.2927989
	0.65	82.97	0.83083	1.1877671		236.2	2.1935	3.1854981
	0.7	81.16	0.80458	1.1525728		226.9	2.15733	3.1174065
	0.75	79.22	0.79965	1.1413354		216.8	2.11148	3.0356828

Table 5.2 Quantity of concrete, reinforcement and cost of container for different value of λ2 and λ_1 =0.7 α=50^0

Capacity (KL)	λ2	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container (lac)	Capacity (KL)	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container (lac)
300	0.2	32.811	0.3415	0.48447	1000	132.6871	1.2659	1.82795
	0.25	31.699	0.3239	0.46117		125.836	1.2022	1.73543
	0.3	30.908	0.3229	0.45777		120.8504	1.2068	1.72618
	0.35	30.330	0.3234	0.45668		121.8994	1.1827	1.70175
	0.4	29.899	0.313	0.44357		124.3927	1.2105	1.7407
	0.45	29.576	0.3318	0.46401		133.4116	1.2034	1.75876
	0.5	29.333	0.3233	0.45359		141.0548	1.1964	1.77294
500	0.2	53.702	0.5196	0.74807	1250	190.0523	1.6926	2.48073
	0.25	51.175	0.5138	0.73419		180.1343	1.6781	2.43548
	0.3	49.330	0.5213	0.73729		172.9295	1.6667	2.40153
	0.35	47.937	0.5157	0.72693		173.1034	1.6682	2.40376
	0.4	46.860	0.5124	0.72003		181.1545	1.6093	2.35991
	0.45	48.782	0.4963	0.70727		191.3672	1.6533	2.43976
	0.5	50.048	0.5467	0.76839		200.0845	1.6839	2.49992
750	0.2	88.739	0.8839	1.26495	1500	250.5416	2.2733	3.31817
	0.25	84.350	0.8243	1.18427		236.8773	2.1869	3.18002
	0.3	81.157	0.8046	1.15257		226.9128	2.1573	3.11741
	0.35	78.755	0.8169	1.15968		236.7901	2.0798	3.05769
	0.4	81.098	0.816	1.16548		245.5014	2.1223	3.13137
	0.45	79.862	0.8083	1.15304		253.2507	2.173	3.21162
	0.5	88.157	0.8778	1.25633		266.5529	2.2271	3.31184

Table 5.3 Quantity of concrete, reinforcement and cost of container for different value of α and λ2=0.3 λ_1 =0.7

Capacity KL	α	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container in lac	Capacity KL	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container in lac
300	35	28.7409	0.355435	0.488545	1000	93.17201	1.271578	1.719798
	40	29.20365	0.353309	0.487463		99.82198	1.238748	1.701656
	45	29.88701	0.337145	0.471017		108.6655	1.190994	1.672864
	50	30.90829	0.322926	0.457769		120.8504	1.206769	1.726183
	55	32.47002	0.309947	0.447503		138.3493	1.255399	1.832368
	60	34.94639	0.311486	0.456438		164.8083	1.299719	1.959623
500	35	41.99098	0.550561	0.749414	1250	129.4937	1.752411	2.37328
	40	43.71413	0.535028	0.736703		139.9523	1.726115	2.373633
	45	46.05342	0.518818	0.725008		153.8313	1.684742	2.366717
	50	49.3304	0.521253	0.737287		172.9295	1.666697	2.40153
	55	54.09622	0.526566	0.757164		200.3463	1.676157	2.491823
	60	61.36443	0.522186	0.773249		241.8319	1.796442	2.749256
750	35	65.25571	0.880233	1.192708	1500	176.4983	2.231715	3.056001
	40	69.05107	0.850311	1.169603		191.0981	2.169519	3.027437
	45	74.12865	0.820228	1.150033		210.4978	2.112708	3.018931
	50	81.15752	0.804575	1.152573		226.9128	2.157333	3.117406
	55	91.28529	0.820364	1.199943		275.693	2.161142	3.263211
	60	106.6266	0.875638	1.307444		334.0363	2.325848	3.620172

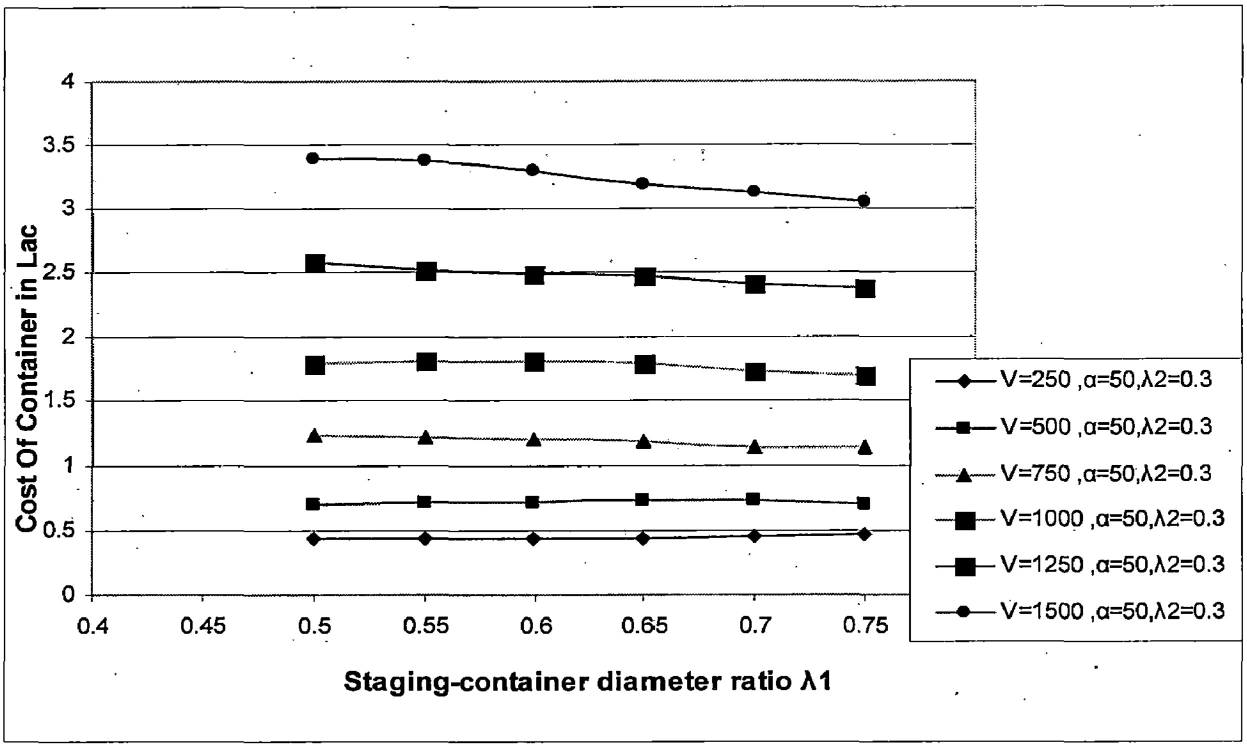

FIG 5.1 VARIATION OF COST OF CONTAINER WITH RATIO $\lambda 1$

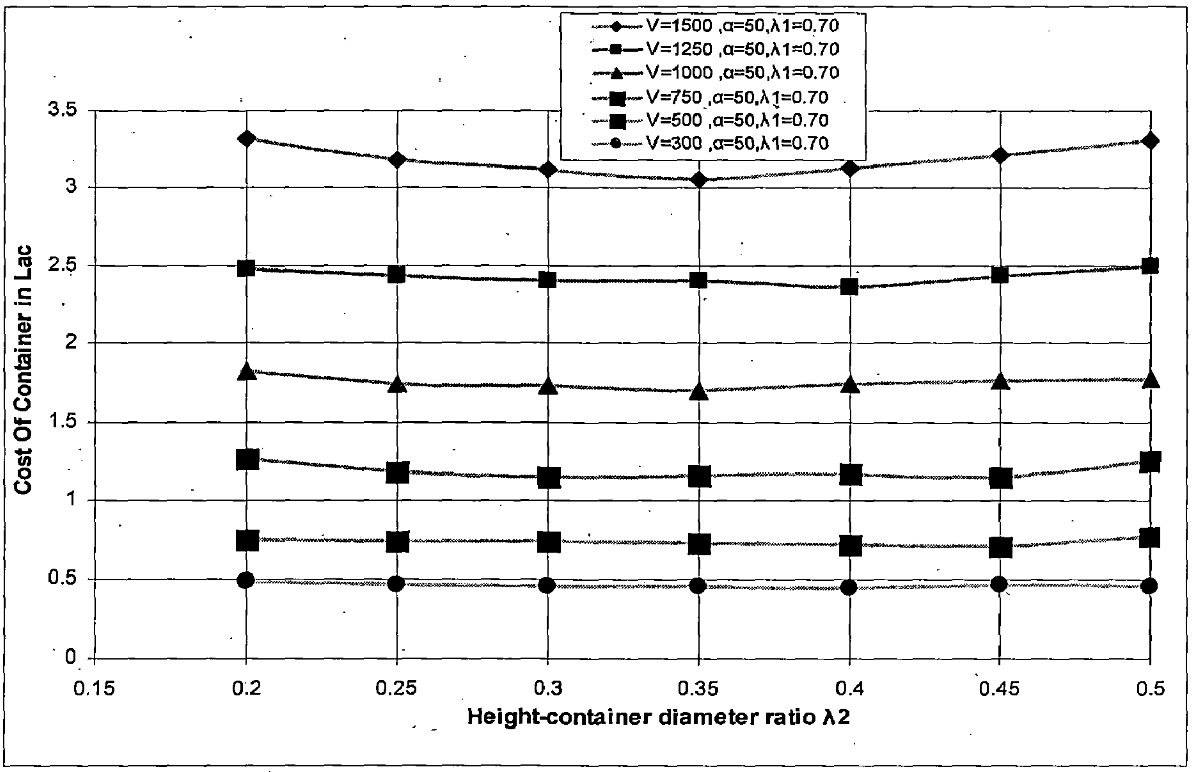

FIG 5.2 VARIATION OF COST OF CONTAINER WITH RATIO λ2

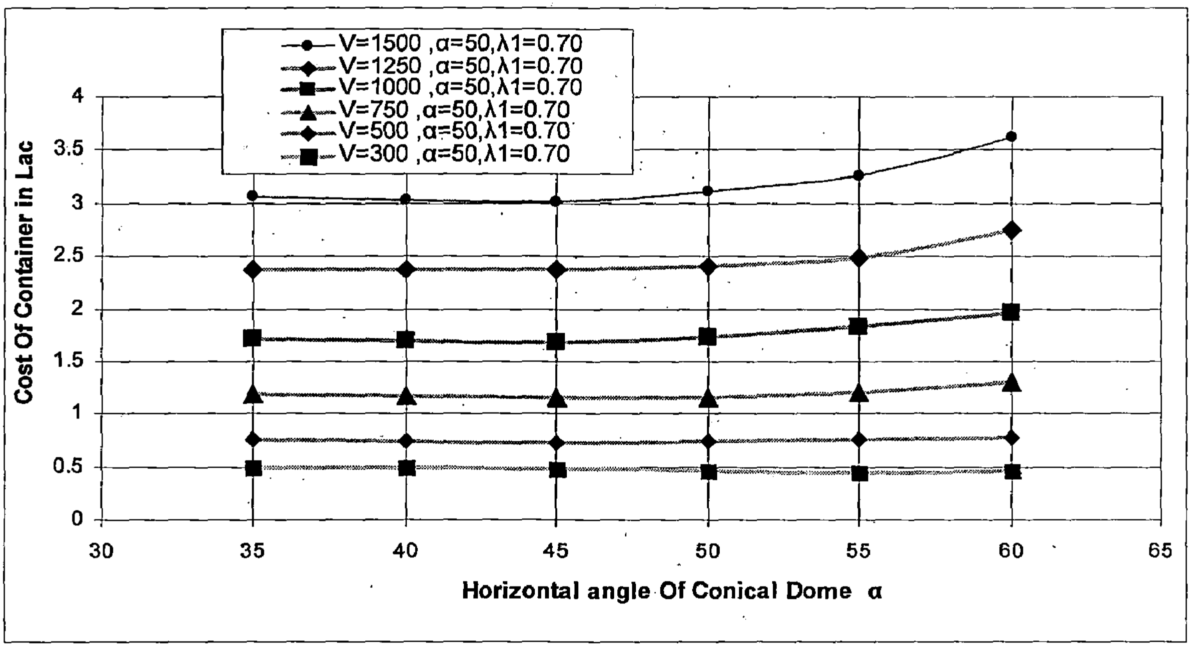

FIG 5.3 VARIATION OF COST OF CONTAINER WITH RATIO α

5.3 STAGING

Obtaining from optimum value of different parameter of container, the design of staging is done by varying the values of basic parameter such as capacity of tank V, height of staging Hs and various wind zones .for economical design we have varied no of columns, column diameter, and minimum no of panels .the design is carried out by limit state approach. The result of cost of staging are tabulated in table 5.2 for six different capacities and for three height of staging, 15m, 18m, 21m.The table 5.4 graphically presented from Fig 5.4 for staging height 15m. The curves clearly indicate the effect of column in the cost of staging.

Table 5.4 Quantity of concrete, reinforcement and cost of staging for different number of column and height of staging, 15m, 18m, and 21m.The container has value of $\alpha=45^0$ $\lambda 2=0.3$ and $\lambda_1=0.7$

Capacity KL	Height of staging Hs	No Of column	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container in lac	Capacity KL	No Of column	Volume of concrete (m^3)	Volume of steel (m^3)	Cost of Container in lac
300	15	10	45.0	1.73	2.57	1000	14	113.5	3.81	5.72
		12	49.6	1.67	2.51		16	116.3	3.63	5.4
		14	54.29	1.69	2.55		18	119.7	3.89	5.8
	18	10	56.03	2.42	3.58		14	136.94	5.45	8.09
		12	61.54	2.59	3.85		16	143.39	5.59	8.32
		14	67.0	2.74	4.07		18	150.9	5.27	7.89
	21	10	68.63	3.31	4.89		14	151.94	7.23	10.66
		12	67.26	3.46	5.08		16	159.31	7.54	11.12
		14	73.77	3.72	5.47		18	168.85	8.05	11.87
500	15	12	65.50	2.28	3.42	1250	20	135.9	4.29	6.462
		14	68.41	2.22	3.34		22	139.7	4.37	6.5817
		16	72.06	2.42	3.63		24	143.5	4.65	6.98
	18	12	72.56	3.09	4.59		20	152.93	6.24	9.26
		14	76.65	3.24	4.80		22	158.4	5.95	8.88
		16	81.48	3.39	5.03		24	168.54	6.41	9.55
	21	12	86.89	4.27	6.28		20	169.89	8.63	12.69
		14	94.94	4.63	6.81		22	177.0	8.73	12.85
		16	98.76	4.93	7.25		24	188.8	9.39	13.81
750	15	14	99.36	3.06	4.62	1500	20	148.8	4.913	7.37
		16	102.9	2.87	4.36		22	149.8	4.64	7.0031
		18	111.38	3.15	4.77		24	158.29	4.98	7.50
	18	14	111.2	4.58	6.80		20	170.6	6.76	10.0
		16	116.56	4.35	6.48		22	168.53	6.59	9.8
		18	126.6	4.77	7.10		24	178.6	7.06	10.49
	21	14	123.1	6.08	8.95		20	182.7	9.19	13.5
		16	130.12	5.92	8.74		22	187.19	9.18	13.5
		18	141.90	6.57	9.7		24	198.9	9.85	14.5

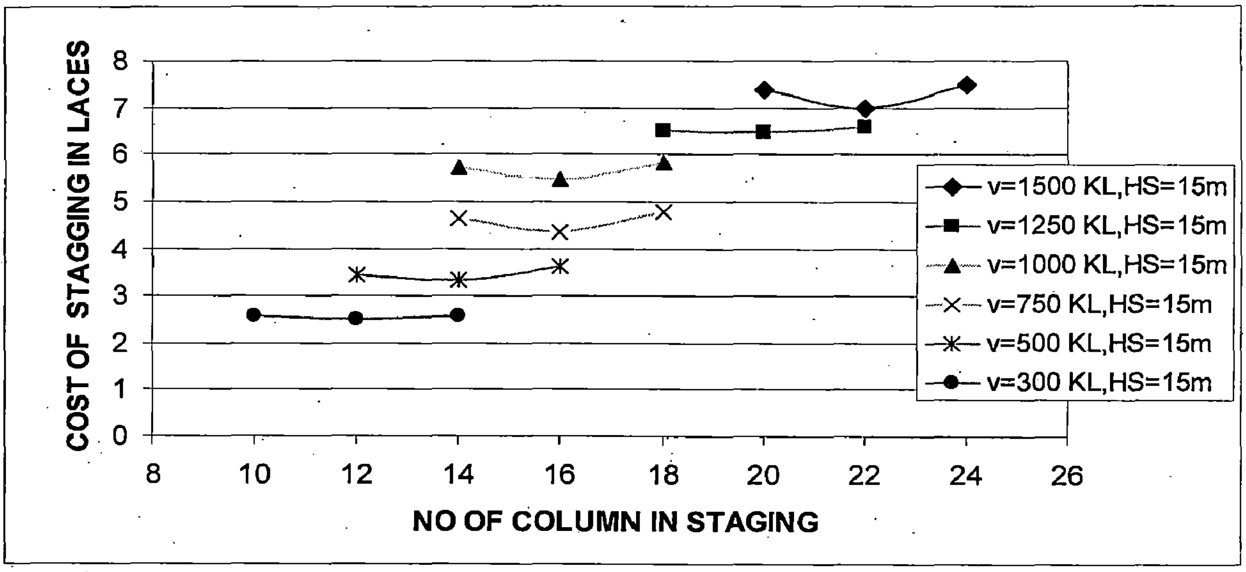

FIG 5.4 VARIATION OF COST OF CONTAINER WITH NO OF COLUMN

CHAPTER 6

DISCUSSION AND CONCLUSION

6.1 DISCUSSION

As the ratio λ1 is increase from 0.5 to 0.75 the cost of container decreases. The effect is more significant in when water storage capacity is more than 750 KL .For lower value of λ1 the reduction in capacity in conical portion of container which enhance the container dimensions and hence the cost of container. The curve Fig 5.1 represents variation of cost of container with value of λ1.

From the result of previous chapter we obtain, as we increase in height –diameter ratio it reduce the cost of top ring beam and bottom dome. But over all cost of container increases due to increasing the cost of cylindrical wall effectively which is shown in Fig 5.2. The cost of container increases rapidly when λ2 value increases from 0.35. For 0.3 to 0.35 the curves for all capacity are nearly a straight line which has very less increment in slope but beyond this slop increases significantly.

Keeping other parameter constant and varying the value of α from 35^0 to 60^0, it is observed that the value of cost of container decreases when α less than 45^0.As the α increased from 45^0, an added capacity is created in conical portion of container which decreases the dimension of cylindrical wall but over all cost of container increase. The change of cost of container with α has been shown in Fig 5.3.

From figure 5.4 it is observed that cost of staging increases as the no of column decreases to a curtain limit because significantly in crease the value of $\frac{P_u}{\sigma_{ch} * D_c^2}$.For safe design as well as economical it is observed that up to 500 KL capacity 12 to 14 column are economical. For capacities between 500 KL and 1000KL 16 column are economical and beyond 1000KL up to 1500KL 20 to 22 are sufficient.

6.2 CONCLUSION

The most economical combination of various parameter for the container are given below

TABLE 6.1 Recommended proportion of container and staging.

Parameter	Optimum value of parameter of container capacity(KL)					
Capacity	300	500	750	1000	1250	1500
λ_1 (D2/D1)	0.5 to 0.6	0.6 t0 0.7	0.7 to 0.75	00.7 to 0.75	0.7 to 0.75	0.7 to 0.75
λ_2 (H1/D1)	0.3 to 0.35	0.3 to 0.35	0.3 to 0.35	0.3 to 0.35	0.35 to 0.4	0.3 to 0.35
Angle(α)	40^0 to 45^0	40^0 to 45^0	40^0 to 45^0	40^0 to 45^0	40^0 to 45^0	40^0 to 45^0
Nc(column)	14	14	16	16	20	22

6.3 SCOPE OF FUTURE STUDY

(a) Excel program can extend to design for foundation. After designing of foundation we can make, a correct combination of parameter for different capacity of intze tank.

(b) Forces obtained at different location in container and staging can be verified by software packages.

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